NO. WW3
MAY 1956

JOURNAL of the

Waterways and Harbors Division

PROCEEDINGS OF THE
AMERICAN SOCIETY
OF CIVIL ENGINEERS

VOLUME 82

THIS JOURNAL

. . . represents an effort by the Society to deliver technical data direct from the authors to the reader with the greatest possible speed. To this end, it has none of the usual editing required in more formal publications.

Readers are invited to submit discussion applying to current papers. For papers in this journal the final date on which a discussion should reach the Manager of Technical Publications appears as a footnote with each paper.

Those who are planning papers or discussions for "Proceedings" will expedite Division and Committee action measurably by first studying "Publication Procedure for Technical Paper" (Proceedings Paper 290). For free copies of this paper—describing style, content, and format—address the Manager, Technical Publications, ASCE.

Reprints from this Journal may be made on condition that the full title of the paper, name of the author, page reference (or paper number), and date of publication by the Society are given. The Society is not responsible for any statement made or opinion expressed in its publications.

This Journal is published by the American Society of Civil Engineers. Publication office at 2500 South State Street, Ann Arbor, Michigan. Editorial and General Offices at 33 West 39 Street, New York 18, New York. $4.00 of a member's dues are applied as a subscription to this Journal.

Journal of the

WATERWAYS AND HARBORS DIVISION

Proceedings of the American Society of Civil Engineers

WATERWAYS AND HARBORS DIVISION, COMMITTEE ON PUBLICATIONS
Evan W. Vaughan, Chairman; John M. Buckley; Joseph M. Caldwell;
Ellsworth I. Davis; Jay V. Hall, Jr.

CONTENTS

May, 1956

Papers

	Number
Developing Port Facilities on Houston's Ship Channel by Frank H. Newnam, Jr.	965
Flood Protection at Wichita and Valley Center, Kan. by Myron W. DeGeer	966
Improving the Gulf Intracoastal Canal in Texas by Willard P. McCrone	967
Design Problems Involved in Protection from Tsunamis by Kenneth Kaplan	968
New Locks on Lower Mississippi and Gulf Coast by Norman R. Moore	969
Control of Rio Grande by United States and Mexico by J. C. Bustamante and J. F. Friedkin	975
Sand By-Passing at a Virginia Tidal Inlet by Thomas J. McDonald and Myron A. Sturgeon	976

Copyright 1956 by the American Society of Civil Engineers.

Paper 965 WW 3

JOURNAL
WATERWAYS AND HARBORS DIVISION
Proceedings of the American Society of Civil Engineers

DEVELOPING PORT FACILITIES ON HOUSTON'S SHIP CHANNEL

Frank H. Newnam Jr.,[1] M. ASCE
(Proc. Paper 965)

SYNOPSIS

In the past virtually all wharves, both public and private, at the Port of Houston have been of the marginal type because of the unusual nature of the Houston Ship Channel. Unlike most deep water ports, the turning basin of the Port of Houston is 50 miles from the open sea (Gulf of Mexico) and was developed by dredging out a narrow, shallow, winding stream known as Buffalo Bayou. As an inducement to industry, a strip of land 2,500 feet deep on each side of the center of the channel was taken into the City of Houston but declared exempt from City taxes. This forced the development of industrial facilities and their private wharves to be as close as possible to the channel and helped to dictate the utilization of marginal wharves.

The property owned by the Port of Houston, on which its present public wharves are located, is relatively shallow in depth, and its wharves have also been of the marginal type. The Port has recently, however, acquired a tract of land of sufficient depth to permit consideration of the construction of pier type wharves and slips. Because of this fact and the desirability of properly planning in advance for the most economical ultimate layout of the new area, a detailed study was made to develop an approved plan of development for this area and the estimated cost of the incremental parts. These studies included visits to other major deep water ports of the nation and conferences with representatives of the various agencies that will utilize the new facilities.

As a result of these studies a table of basic criteria and dimensions was adopted for marginal type and pier type facilities. After careful study of several possible schemes of layout the marginal wharf type of development was recommended as the most satisfactory for the site conditions.

Research on the subject at the start of the study revealed a paucity of information and basic criteria on this subject. It is believed, therefore, that the criteria developed, factors considered and conclusions reached will be of value to others who may be considering an expansion of port facilities.

Note: Discussion open until October 1, 1956. Paper 965 is part of the copyrighted Journal of the Waterways and Harbors Division of the American Society of Civil Engineers, Vol. 82, No. WW 3, May, 1956.

1. Partner, Lockwood, Andrews & Newman, Cons. Engrs., Houston, Tex.

INTRODUCTION

The Houston Ship Channel is probably the most unusual deep water port in the nation. As shown in Figure 1, the harbor frontage commonly referred to as the Port of Houston extends on both sides of the Houston Ship Channel for a distance of 25 miles. The turning basin, at the head of deep water navigation, is 50 miles from the Gulf of Mexico and 58-1/2 miles from the end of the jetties. The upper portion of this channel has a bottom width of 250 feet and a depth of 36 feet, but a proposed ultimate project depth of 40 feet and channel width of 300 feet. The history of the ship channel is interesting because local interests paid one-half of the initial cost of construction in a field where the Federal Government has traditionally assumed all of this burden. For the past few years the Houston Ship Channel has ranked second of the deep water ports of the nation in the amount of tonnage handled.

In order to attract industry, a strip of land 2,500 feet deep on each side of the center of the channel was taken into the city of Houston but declared exempt from city taxes. This forced the development of industrial facilities to be as close as possible to the channel and helped to dictate the utilization of marginal type wharves.

While the Harris County-Houston Ship Channel Navigation District, usually referred to as the Port of Houston, owns and operates considerable port facilities, these facilities are a small percentage of the total along the ship channel. The tracts of land owned by the District are relatively shallow in depth and therefore all wharves constructed by it to date have also been of the marginal type.

A few years ago, however, the Port of Houston purchased a tract of land southeast of the public grain elevator and less than one mile downstream from the turning basin, which, together with two small adjacent tracts planned for purchase, contains some 328 acres and sufficient depth to permit, for virtually the first time, consideration of the construction of slips. This property has a total channel frontage of 9,000 feet and an average depth of some 1,600 feet. The tract is easily served by adjacent roads, utilities and railroads.

An important factor in the consideration of its development is the topography of the land which is at elevation plus 25 to 30 feet along the channel and rises to an elevation of approximately plus 40 feet at the rear. The disposal of spoil from the area will present an item of considerable expense inasmuch as the closest dumping grounds for hydraulic spoil disposal are more than one mile away, and considerably further for spoil that is hauled away in vehicles. It might be added parenthetically that the problem of providing spoil disposal areas for maintenance dredging, and widening and deepening operations, is rapidly becoming a serious problem for the upper section of the ship channel.

The 328 acre tract of land is the last tract of any size that is available to the Port of Houston near the turning basin for the expansion of its port facilities. Since the tract has sufficient depth to permit the consideration of "finger pier" or "slip" type of construction, as well as the "marginal wharf" type, it was decided that thorough engineering studies be made in order to determine in advance the most advantageous layout of wharf and dock facilities for the area, including all supporting facilities and operations such as shipping, trucking, railroads, handling and storage of cargo, utilities and roads. It was also desired, of course, to obtain preliminary estimates of construction costs of the facilities included in each of the schemes considered.

The Port engaged the two Houston consulting engineering firms of Robert J. Cummins and Lockwood & Andrews to jointly make this study. These engineering studies and recommendations will make it possible for the Port of Houston to adopt a full layout plan of the area so that its development by stages will eventually produce the best layout of facilities for the entire area.

While numerous layouts were considered, essentially the comparison was between the "marginal wharf" type and the "finger pier" type of marine terminal facilities, and only these two basic schemes are discussed herein. One basic structural design of the wharf and pier substructure; namely, the design used for Wharf No. 8, the newest wharf at the Port, was selected and used in all schemes for the sole purpose of developing comparative cost data, based on 1955 prices. It was recognized that detailed analyses and probably different designs would have to be made for each separate facility at the time that contract plans and specifications are prepared.

Extent of Investigations and Studies

The schemes for development of the area, and the recommendations contained in this study, evolved from:

a) A study of the principal features of the site, including location, topography, utilities, transportation facilities, and probable future usage;

b) Preliminary development of several basic types of layout that could be considered for the site;

c) Scheduled conferences and consultations with port officials of the Port of Houston and port officials of other major ports, railroad officials, shippers, truckers, and other interests pertinent to the operation of the facilities and handling of cargo, etc.;

d) Visits to and observations of the latest facilities of several major ports of the United States, including San Francisco, New York, Seattle, Mobile, Long Beach, Los Angeles, and New Orleans;

e) A study of written material published by port officials, surveys by various organizations, and governmental publications;

f) An overall study of the various schemes developed for the utilization of the new area, comparative cost data, and the particular anticipated needs of the Port of Houston.

Information from Inspection of Other Major Ports

General

The major ports of San Francisco, New York, Long Beach, Los Angeles, New Orleans, Seattle, and Mobile were visited for the purpose of observing the most recent and modern layouts of marine terminal facilities, of both marginal wharf and finger pier type, and to discuss such layouts with, and to receive the recommendations of, key port officials (both operators and engineers) of each port authority. Basic dimensions, arrangement of facilities, amount of storage and handling space desirable, and other such principal features of both the finger and marginal type piers were discussed. The advantages and disadvantages, from an operation standpoint, of the use of finger type piers and marginal type wharves or combinations of the two types, were discussed and recommendations were sought concerning the best utilization and development of the property included in this study.

The information gained in observing the new wharf facilities at each of these major ports has served as an invaluable guide and "check" in these studies. Likewise, the observing of older and outmoded installations has been beneficial in that the faults and deficiencies of such layouts or arrangements stressed by the port officials serve as adequate warning in the planning of similar type installations. In weighing the information and recommendations received, the basic differences between the ports observed and the Port of Houston (such as types of cargo handled, arrangement and size of waterways, configuration of waterfront, land space, access to waterfront, types of land transportation principally used in handling cargo, etc.) were considered in preparing the layouts of wharves and other facilities for the particular land involved in this study.

San Francisco

The marine terminal facilities serving the Port of San Francisco are virtually all of the pier type located along the west shore line of San Francisco Bay. This Bay with its vast area of deep water provides ample space for anchorages and the proper handling and turning of vessels, and therefore is ideally suited for finger type piers. Since this is one of the older ports of the nation, the majority of the piers were designed for the days when virtually all of the land shipment was by rail. At the present time such shipments are approximately 25% by rail and 75% by truck. New facilities at this port, such as the Mission Rock Terminal, make full provision for a large paved area inside of the pier for the maneuvering, parking and docking of a large number of trucks.

It was recommended that piers be only one ship's length, 800 feet to 850 feet long, because of the difficulty of berthing the inside ships for piers that are two ships in length. Where the latter are used, however, they should be 1,200 to 1,400 feet in length. In either case, piers should be wide enough to permit a ship to be berthed across the end of the pier. Other recommendations included:

a) Width of slips, one ship length: 225 feet minimum, 300 feet desirable;
b) Width of transit shed: 200 feet desirable;
c) Width of paved area between back aprons: 125 feet minimum;
d) No railroad trackage in transit sheds;
e) Dock height for trucks: 48 inches desirable;
f) Width of rear apron: 15 feet.

New York

New York is the largest and one of the oldest ports of the nation, has a variety of wharf types, but the finger pier type of construction predominates. For all the port facilities there is ample area of water in front of the wharves to permit the maneuvering of vessels during docking operations without interferring with the other water traffic.

One of the newest facilities, and apparently considered in the area to be one of the most satisfactory, is Port Newark located on a large artificial channel connecting into the west shore of Newark Bay. This facility is of the marginal wharf type and is considered to be more efficient and satisfactory from the standpoint of operation than any of the finger pier type of wharves. Two matters of interest on this facility are (1) the offices located in each

shed opposite each berth at a second story level, which permits full view of all operations and provides storage space under the offices, and (2) special cargo rooms, with heat, for valuable cargo storage in the sheds.

Some 80 to 90 percent of land shipments are by truck, and a 100 foot wide area is paved behind the rear apron of the transit sheds. The Port Manager also emphasized the advantage of having large paved areas behind and separated from the shed area which can be used for open storage and transhandling of items that are not fast moving.

The officials of the Port of New York Authority, as in the case of the Port of San Francisco, felt that the ideal width for a transit shed was 200 feet. They also felt that a minimum distance of 900 feet beyond the end of piers was needed for the maneuvering of vessels in docking for this type of facility. This 900 feet would be exclusive of any space required for the passage of other vessels.

Other recommendations included:

a) Transit shed area per berth: minimum of 90,000 square feet for Class A berths; 45,000 to 90,000 square feet for Class B berths;

b) Width of slips: 300 feet minimum if at right angles to the channel; otherwise wider;

c) Width of front apron: 50 feet;

d) Width of rear apron: 17 feet with an 18 foot canopy;

e) Dock height for trucks: 48 inches;

f) Lighting in transit sheds: Industrial type fluorescent lighting at 20 foot centers providing 8 foot candles in warehouse area, plus transparent sheets in roof for daylight help.

Long Beach

The Port of Long Beach is essentially a man made harbor consisting of an inner and outer harbor area and a large number of modern facilities constructed by the Port of Long Beach and the U. S. Navy. Although both pier and marginal wharves are operated, Port officials expressed their belief that marginal are most desirable and economical to operate. Here again the new transit sheds are 200 feet in width and a clear height of 20 feet was desirable. The wharf apron is 50 feet in width with two railroad tracks, 9 and 22 feet from the fender line. There are two spotting tracks and one running track behind the sheds.

There is a 300 foot open wharf and storage area between sheds, which ramps from wharf apron level at the front to track level behind the shed, and the wharf lead tracks run up on this ramp grade. This scheme presents an easy truck and railroad car approach to the wharf apron, and apparently does not present any handicap in handling cargo on this open storage area.

Los Angeles

This port also has both the finger pier and marginal type wharves located at San Pedro Bay which provides ample space for the maneuvering of vessels. Front apron widths were generally 35 to 37 feet, rear aprons 16 feet wide with 16 feet canopies, and transit shed widths 200 feet. Paved areas 100 to 150 feet wide were provided behind the transit sheds for the maneuvering and docking of trucks. The open wharf between sheds is ramped up from truck and track level behind the shed to the wharf apron level.

The piers and slips on terminal island fronting the main channel are built diagonally to the channel with slips 300 feet in width and 100 feet in length and piers approximately 450 feet wide. These are considered ideally suited for the operational needs of the railroads, which handle a majority of the tonnage, with the railroad classification and storage yard located immediately behind the end of the slip.

New Orleans

The marine terminal facilities of the Port of New Orleans are located along the Mississippi River in Louisiana and consist generally of marginal type wharves. The Mississippi River is sufficiently wide to permit the safe handling and berthing of ships at piers, but the developed area adjacent to the river in New Orleans restrict the depth of land available for such terminal facilities.

Many of the marine facilities at New Orleans are quite old and are outdated, and because of the extensive use of manual labor in handling cargo in this area, many of the transit sheds are 100 feet or less in width. There is a minimum of railroad tracks and very little yard space, as a major part of the cargo is handled by trucks.

The Port of New Orleans is correcting many of its unsatisfactory conditions in constructing the new Napoleon Avenue Wharf which will have a wharf apron 47 feet wide, a transit shed 200 feet wide with a 19 feet clear height, and a rear apron 30 feet wide with a 15 foot canopy.

Mobile

The most modern facility at this port is a marginal wharf type marine terminal with a 30 foot wide apron, a 200 foot wide transit shed with a clear height of 20 feet. As in the case of the Port of Newark, there is an office at second story level for each berth. The loading apron at the back of the shed is 25 feet wide and differs from most of the other new facilities in that no permanent cover is provided.

Seattle

Seattle has two harbors, one a fresh water harbor serving the lakes, and the other a salt water harbor with the principal facilities located in Elliott Bay. This is a spacious, deep sheltered body of water, free of hazards to navigation and ideally suited to the pier type layout, and the majority of facilities are of this type. The observations made in this visit, and comments and drawings furnished by port officials, have served to add useful data for studies of pier type layouts.

Conferences with Using Agencies

General

Conferences were scheduled with railroad officials, operators of steamship and stevedoring companies and representatives of trucking interests. A separate conference was scheduled with each group, since each interest had its own operational requirements and it was the main purpose of these conferences to obtain a frank discussion of the basic needs and operation requirements of each group and to incorporate their recommendations and

proposals, insofar as possible, in the layout of the new terminal facilities. Since these interests are necessarily conflicting in many respects, the final layout and arrangement must necessarily be a compromise.

Steamship and Stevedoring Companies

All steamship and stevedoring company representatives, without qualification, favored the marginal wharf type layout. The use of this type facilitates the berthing of ships and is considered safer in general for the handling and berthing of ships in the comparatively small water front area provided by the Houston Ship Channel. They felt that the pier type construction would result in slow, costly and hazardous berthing operations on the narrow channel and that any type of finger pier layout should provide a minimum distance of 900 feet from the end of the pier to the far edge of the ship channel in order that ships may safely enter and leave slips. This 900 feet does not include any room for the passing of other vessels, which fact would delay traffic on the ship channel.

These operators agree generally upon the following minimum basic dimensions and arrangements of facilities for the finger pier type of layout:

a) Width of slip — 300 feet

b) Slip length (1 ship length) — 800 feet

c) Width of warehouse (transit shed) — 200 feet

d) Overhead clearance in warehouse — 18 feet

e) Width of paved loading area between warehouses (transit sheds) — 150 feet

f) Width of front apron — 50 feet

g) Width of rear loading platform (rear loading platform should be covered) — 20 feet

h) Number of railroad tracks on front apron — 2

i) Number of tracks back of each warehouse — 2

j) Double RR crossovers, front apron — 1 at center of each berth

k) Single RR crossovers, front apron — 1 at each end of berth

l) Front apron tracks should be far enough apart to permit operation of crane on second track.

The controlling dimensions and basic arrangements agreed upon for marginal type wharf layouts are the same as above except as follows:

a) Width of paved loading area behind warehouse (transit shed) — 100 feet

b) Number of railroad tracks on front apron (need running track along with two spotting tracks as tracks serve a greater number of berths than in the finger pier type layout) — 3

c) Number of railroad tracks at rear of warehouse for marginal type wharf could be a minimum of 2 tracks, but preference for 3 tracks expressed by the entire group — 2 or 3

Railroad Companies

In the conferences with representatives of the various operating railroads, these representatives all agreed that the finger type layout is most desirable from the standpoint of the operation requirements of the railroads, as this layout affects the most economical and simplest switching and handling of cars within the wharf area and makes it possible to reduce the time required in moving cars from the spotting tracks at the loading aprons to the car storage yards.

These representatives agreed to the following factors as being desirable and applicable to both the marginal and finger pier type construction:

a) A maximum grade of 1% for connecting tracks.

b) A minimum curve of 12° 30', except in a few special critical areas where 15% curves will be satisfactory.

c) A minimum spacing between adjacent tracks of 13 feet, although in order to provide double crossovers the limited space available at the front apron may require an increased spacing of approximately 19 feet.

d) A storage yard capable of 500 cars for the entire area with storage track lengths of approximately 50 car lengths.

For marginal type wharf, three tracks, two spotting tracks and one running track, would be desirable for the front apron with a single crossover at the end of each berth extended to the running track adjacent to the transit shed. An outlet from the running track to the storage yard should be located at the end of each warehouse.

At the rear of the rear apron there should be two spotting tracks, with the first track 8-1/2 feet from the rear apron edge, and the second track 13 feet on centers behind and the rearmost (running) track approximately 100 feet from the rear apron edge to allow for truck loading and unloading at the rear apron. Crossovers for these tracks should be provided at the end of each warehouse. The independent switching of cars for each berth could also be obtained by having independent "dead-end" spotting tracks at each berth, with lead-ins from the running track at one end of each berth.

Trucking Companies

Representatives of the trucking companies had no particular preference between the marginal wharf and finger pier type layout provided that a paved area 100 feet deep is provided behind the rear apron for marginal type wharves and a 150 foot paved loading area is provided between back aprons of warehouses for finger piers.

They recommended that the rear aprons of warehouses be 20 feet in width covered for the full width, with a platform height of 48 inches.

This group recommended driving lanes inside the transit sheds to permit the handling of large volumes of packaged merchandise, such as sugar, flour and salt. Clear span inside should be 18 feet minimum with a few 18 foot high doors, although a 10 foot or 12 foot door height was considered generally satisfactory for truck handling.

This group also recommended that one large (150 ton) and one small loading device be provided for loading and unloading cargo in the open storage area. They also requested ample space for parking and turning of trucks in the parking and holding areas. They pointed out the increasingly higher percentage of land shipments by trucks instead of by rail from major ports of the country.

Other References

Numerous other references were studied which furnished useful data concerning desirable basic dimensions and layout of marine terminal facilities. These included Governmental publications such as the Port Series published by the Corps of Engineers, Department of the Army, and the U. S. Maritime Commission, and the annual report of the Port Development and Construction Committee of the American Association of Port Authorities. This Committee sent questionnaires concerning basic dimensions of terminal facilities to various ports and a second set of questionnaires to steamship and stevedore companies on the Pacific Coast for use in future development at the Port of Long Beach, California. The information contained from these questionnaires is summarized in the following table.

SUMMARY OF ANSWERS TO QUESTIONNAIRE FORWARDED TO STEAMSHIP AND STEVEDORING COMPANIES REGARDING PROPOSED PORT OF LONG BEACH HARBOR FACILITIES

Harbor Facility	Min.	Max.	Avg.
Depth, at low water, of main channel to outer and inner harbor	32'	40'	40'
Bottom width of main channel to outer and inner harbor	300'	2000'	600'
Depth of slips alongside marine terminals	32'	40'	35'
Width of slips between piers of approximately 2000' length	300'	500'	400'
Length of berth for largest ocean going vessel	500'	1200'	850'
Length of berth for average size ocean going vessel	450'	600'	500'
. . .			
Width of transit shed	140'	250'	200'
Vertical clearance between floor and lower chord of truss in shed	16'	25'	20'
Longitudinal widths between columns	30'	75'	40'
Water-side door openings continuous or every other panel	continuous	every other panel	every other panel
Land-side door openings continuous or every other panel	continuous	every other panel	every other panel
Vertical clearance of waterside door openings	12'	20'	16'
Vertical clearance of landside door openings	12'	18'	14'
Widths of waterside door openings	12'	22'	18'
Widths of landside door openings	12'	20'	16'
Vertical clearance of door openings in end of shed	14'	25'	20'

Harbor Facility	Min.	Max.	Avg.
Width of door openings in end of shed (2 doors)	14'	20'	18'
One story or two story transit sheds favored	one	one	one
. . .			
Width of roadway space between sheds (including track facilities)	150'	500'	200'
Width of rear loading platform of transit shed	10'	15'	12'
Number of railroad tracks on apron wharf of shed	2	2	2
Number of spot tracks in rear of transit shed	2	3	2
. . .			

Basic Dimensions and Criteria Adopted

After reviewing and analysing all of the data previously described, it was deemed necessary to adopt certain standards for basic dimensions, arrangement of facilities, and other criteria that is common to the various wharf facilities so as to compare alternate schemes on an equivalent basis. While an attempt was made to select the dimensions and criteria that appeared at this time to be the most satisfactory for these facilities, it was emphasized that each separate facility should receive more thorough and careful analysis during the preparation of contract plans and specifications and that many of these basic dimensions can be varied to suit the particular facility so long as it does not interfere with the ultimate coordinated development of the layout that is finally selected and adopted for this area.

For the marginal wharf type of development, a pattern was adopted which provided two berths with sheds followed by one open wharf. Since each berth was 600 feet long, and the shed allocated to each covered berth only 500 feet long, the spacing of sheds along the wharf required considerable study, and first consideration was given to centering the sheds on each berth. This left an open space between sheds that apparently would have little use, therefore, it was deemed more desirable to place the two 500 foot sheds together and make one shed 1,000 feet long, with firewalls. This arrangement leaves the open wharf between sheds 800 feet long on the marginal type facility. It should be mentioned that the arrangement of two berths with sheds followed by one berth with open wharf not only provides an economical balance between the relative needs for open storage and sheds, but also permits railroad entrance to the front tracks at each third berth.

In adopting criteria and dimensions for transit sheds, no provisions were made for truck and railroad car traffic inside the shed as part of the normal cargo handling operation because of the problem of pilferage. However, it is possible under the schemes presented herein for trucks to enter the shed from the rear side or the ends.

The visits to other major ports of the United States have indicated the wide-spread use of, and the urgent need for, paved areas behind the wharf facilities for personnel parking, holding yards for trucks, and open cargo storage. At the time of building new marine terminal facilities, thorough studies should be made to determine the quantity and location of such paved areas. Although the exact usage cannot be defined at this time, the need for

such paved areas has been recognized in this study and an amount has been included in the cost estimate for such paved areas located along the access road paralleling the waterfront and for a truck holding yard.

The design water depth at the fender line was selected as elevation minus 40 because this is the project depth for the Houston Ship Channel. The elevation of the wharf apron at adjoining Wharf 16 is elevation plus 18. However, the two newest wharves in the turning basin area are at elevation plus 21. Elevation plus 20 was selected as the apron height in the new area because a two foot differential can be tied into Wharf 16 without too long a transition section. The reasons for selecting elevation plus 20 or 21 for aprons at this port are:

a) The high ground on which the wharves must be built makes a high elevation desirable from the standpoint of economy in the removal of earth and connection with adjacent railroads;

b) The steamship and stevedoring companies prefer the high elevation;

c) While infrequent, the combination of high tides from tropical hurricanes, plus heavy rainfall in Buffalo Bayou and San Jacinto River (which usually accompanies tropical hurricanes) can produce extremely high water levels in the ship channel.

The basic dimensions and arrangements adopted are listed in Table 1. Figure 2 shows the principal dimensions and elevations for a typical section of wharf and shed under the marginal wharf plan of development. Figure 3 shows the principal dimensions and elevations for slip, shed and area between sheds on a typical cross section of the finger pier plan of development.

At the beginning of the studies every effort was made to find previous reports, articles, or other publications that furnished criteria and basic dimensions for the development of marginal type and/or pier type wharf facilities. Remarkably little information was found to be published on this subject. Therefore it is believed that these criteria, even though they are considerably modified for other locations and conditions, will be of value to others confronted with a similar problem of planning the layout of terminal facilities.

Analysis of the Two Basic Layouts Considered

General

After a preliminary study of various possible layouts of facilities, four schemes were prepared for the best utilization and development of marine terminal and supporting facilities in the area under study. Of the four schemes, one was entirely marginal wharves, one entirely piers and slips, and two schemes were combinations of the two. It was found that the decision rested between the two basic types of layouts, therefore only the two basic schemes are considered herein. Figure 4 shows the marginal type wharf plan of development and Figure 5 shows the finger pier type wharf plan of development. Each scheme has some advantages and some disadvantages, therefore the final choice should be based upon a careful analysis of these factors.

Marginal Type Wharf Plan of Development

The best layout utilizing marginal type wharves is shown on Figure 4, wherein transit sheds and wharf aprons parallel the ship channel. This plan

TABLE 1

BASIC DIMENSIONS AND CRITERIA ADOPTED

ITEM	DIMENSION OR CRITERIA
1. Wharf Apron	
Height above M.L.W.	20'
Width	50'
2. Transit Shed	
Width	200'
Length per Berth	500'
Clear Height (Bottom of roof truss to floor)	20'
Width of Rear Loading Platform	20'
Width of Canopy, Rear Loading Platform (extended to centerline of first track)	28'-6"
3. Width of Paved Loading Area Back of Shed	
Marginal Wharves	100'
Finger Piers (distance between rear loading platforms)	150'
4. Length per Berth (Marginal Type)	600'
5. Single Slip Length (Pier Type)	800'
6. Width of Slip (Pier Type)	300'
7. Railroads	
General	
Minimum Track Spacing	13'
Maximum Degree of Curvature (where economically possible)	12° 30'
Maximum Grade (where economically possible)	1.0 percent
Storage Yard Capacity	500 cars
Length of Storage Yard	50 car length
Marginal Type	
Number of Tracks, Wharf Apron	3
Number of Tracks, Behind Shed	3
Double RR Crossovers, Wharf Apron (between spotting tracks)	1 at center of each berth
Single RR Crossovers, Wharf Apron (between spotting tracks and extended to running track)	1 at each end of berth
Single RR Crossovers, Behind shed (independent switching of cars at each berth)	1 at each end of shed
Pier Type	
Number of Tracks, Wharf Apron	2
Number of Tracks, Behind Shed	2
Single RR Crossovers, Behind Shed (independent switching of cars at each berth, where more than one berth length of pier)	1 centered between berths
Double RR Crossovers, Wharf Apron	1 at center of each berth
8. Roadways	
Main Road Width	44'
Access Road Width	24'
9. Design Depth of Water at Fender Line	-40'

provides for 15 new berths; 10 first class covered steamship berths and 5 open wharves. Because of the small scale the railroads are not shown on Figure 4, however, the location of railroads in relation to the front apron and rear apron are shown in cross-section on Figure 2. From the running track in the rear, and from the classification yard, railroad tracks connect with the tracks on the front of the apron by crossing each open wharf. There are two tracks entering across each open wharf, one passing to the transit shed on the south and the other passing to the front of the transit shed on the north of the open wharf.

The marginal wharf type layout is considered safer, and best suited for navigation operations on the type of waterfront provided by the Houston Ship Channel, consisting of relatively narrow waterway on a meandering alignment. This scheme is preferred by both steamship and stevedoring companies and is considered to be the best scheme for the safest and also most economical handling and berthing of ships.

This layout required less depth of land area and far less spoil of excavated materials than the finger pier type of layout. Quantities of excavated and dredged materials to be spoiled are approximately one-sixteenth the quantities required in the scheme using piers, as hereinafter reflected in the cost estimate which indicates the total costs of excavation and spoil of such materials. In addition to any cost savings, the selection of this scheme would mean that considerably less spoil would be placed in the dumping grounds across Clinton Drive. Such dumping grounds are needed for long term maintenance dredging in the ship channel, and since all areas around the turning basin are becoming highly developed and industrialized, such spoil areas are becoming increasingly harder to obtain. A further advantage of this scheme is that no land is lost to dredging behind the harbor line and the depth of land not used for waterfront facilities may be used for open storage, personnel parking, truck holding yards, storage warehouses and for other needs which may arise in the future.

There are no slips or waterways behind the harbor line in this scheme as in the other. Therefore the Federal Government will maintain the ship channel at project depth, and up to within 50 feet of the fender line of the wharves.

The unbroken waterfront apron of this scheme facilitates the moving of cargo in emergency operations from one shed or berth to another shed or to a waiting ship up or down the channel. This type of waterfront also provides ready access and ample space for fire boats during fires, and fires at one berth would be more remote from, and therefore less hazardous to, other berths than in schemes utilizing pier type construction.

The 100 foot wide paved area behind transit sheds provides ample room for the maneuvering and parking of trucks, and trucks have access to the full length and the ends of every transit shed. Since the running track behind sheds serves the entire waterfront, the rail traffic through the paved area is heavier in this scheme than the other; however, there are fewer tracks crossing the access road which parallels the waterfront than in the pier type layouts.

The railroads do not consider this scheme as desirable as the finger pier type facility, inasmuch as they consider the pier type facility provides the better scheme for the most economical handling and switching of cars between the wharf apron and the storage and classification yard.

Fewer lineal feet of bulkhead per berth are required for wharves of the marginal type than in the pier and slip type, since end treatment of slips and

extra length of slips over berthing distance are required in pier and slip type of wharf facilities.

This type layout has one-fourth less berths, since the marginal wharf layout parallels the channel and does not utilize any land depth for berthing space.

Finger Pier Type Wharf Plan of Development

Figure 5 shows the apparent best layout utilizing this type of pier, which has the major advantage of providing the greatest number of berths (20) for the area, as compared to 15 berths for marginal type layout. In addition it would be physically possible to add additional berths in the future by increasing the length of some of the slips and re-arranging tracks and paving. Here again, for the sake of clarity on the small scale map, the railroad lines themselves are not shown on Figure 5. A connection is made between classification yard and the running track extending the full length of the area to the tracks on the front apron and behind the rear platform, which tracks are shown on the typical section in Figure 3. There are no railroads inside of the transit sheds or at the ends of the bulkheads.

Nineteen covered berths provide a gross shed area of 1,900,000 square feet as compared to almost half that amount in the marginal wharf type layout; however, this scheme has only one open wharf and possibly more open storage space would be found to be desirable during actual development. The cost per berth in this scheme is considerably higher, since a greater length of bulkhead is required per berth than in the marginal wharf construction.

Concerning navigation and the berthing of ships, the advantages enumerated above for the marginal wharves are reversed in this layout, in that the relatively narrow and meandering ship channel does not provide the open waterfront space that usually accompanies finger pier layouts. At least 900 feet is needed from the end of the piers to the far edge of the channel for the safe berthing of ships, with no allowance for passing of other ships in the channel. This arrangement would hinder traffic in the ship channel.

The layout of berths at an angle to the channel, and conformance with the clear distance requirements discussed above, results in 106 acres of land lost to dredging behind the harbor line and considerable loss of land depth needed for open storage and parking areas in the rear. The slips in this scheme are waterways behind the harbor line and outside of the scope of dredging operations now being performed by the Federal Government in maintaining the ship channel at project depth; therefore, the dredging of the slips, as well as the area between the channel and piers, would have to be a maintenance expense of the Port of Houston. The land required by the piers at the downstream end of the property results in the use of the total depth of land at that end of the area under study. In addition to the loss of land depth, this scheme has the greatest amount of dredged material to be hauled over the relatively long haul distances and spoiled in the dumping grounds now used for long term maintenance dredging spoils.

In this scheme, there are no tracks on the wharf apron and behind sheds which serve the entire waterfront area and therefore there would be less rail traffic in these paved areas; however this scheme has the greatest number of tracks crossing the access road which parallels and serves the waterfront area.

Railroad officials definitely prefer this scheme and it is generally

considered that the pier layout is best suited for the most economical switching of cars in the wharf area.

Cost Comparison

Preliminary cost estimates have been developed for each of the two layouts under study and these cost data and other information used in evaluating and comparing the cost totals is shown in Table 2. As described heretofore under "Purpose and Scope," one basic structural design of the wharf and pier structures was selected and used in all schemes in order to develop the comparative cost data, based on 1955 prices, and is not intended to indicate proposed structural design. Preliminary structural design studies were not within the scope of this study and should be based on thorough field investigations of soil and site conditions for each wharf site whenever contract plans are prepared.

The estimated cost of each scheme does not include amounts required for land acquisition, legal fees, bond and other financing costs. The total cost of each scheme is based on the consideration that any scheme adopted would undoubtedly be developed in increments over a period of years and is not intended to represent the estimated cost of constructing all the facilities of a scheme at one time.

As shown in Table 2, the marginal wharf layout has the lower total cost. Items which vary with the arrangement of facilities in each scheme, such as number of berths, number of open wharves, and total gross shed area, must necessarily be included as factors in evaluating the total estimated cost for each scheme. These factors of evaluation are included in Table 2 immediately below the total cost figures.

Using these evaluation factors, the total cost per scheme was reduced to cost per berth, with all marine terminal facilities included, and the cost per berth of the marginal wharf layout is the lower.

Since the gross shed area per scheme varies from 1,000,000 to 1,900,000 square feet, the truest comparison of cost is reflected in the last item which shows "cost per berth, less sheds," and the cost per berth is again lower for the marginal wharf type layout.

The fact that the marginal wharf layout is lower in cost per berth, even with transit sheds excluded from cost figures, is attributed largely to the excess amount of bulkhead required per berth in the pier and slip type layout, as compared to the marginal type layout. The excavating, dredging, and spoil items also increase the unit cost per berth of the pier type construction above the unit cost per berth of the marginal wharf type construction.

CONCLUSIONS

These conclusions evolved from studies and comparisons of all schemes considered for development of the area, the two basic type schemes prepared for the particular area under study using basic criteria and dimensions adopted after numerous conferences with Houston operators and port authorities, visits to other major ports in the United States, studies of bulletins and reports published by various port authorities, and after an overall study of the information thus obtained in order to apply it to the area being considered. The conclusions are as follows:

TABLE 2

COST ESTIMATE COMPARISON

Item	Description	Marginal Wharf Type Layout	Finger Pier Type Layout
1	Wharf structure from front fender through 2nd rear RR track, including transit shed floor and apron floor	$21,175,000	$39,600,000
2	Transit shed superstructure, including electrical, plumbing, etc. inside shed (no sprinkling)	3,500,000	6,650,000
3	All dredging to front of fender line, including spoil disposal	125,000	1,656,000
4	Land excavation and spoil disposal not included in Items 1 and 3	720,500	953,000
5	Access road, concrete paving and drainage necessary thereto	242,324	231,075
6	Overpass	250,000	250,000
7	Underpass	200,000	200,000
8	Additional paved areas not included in Item 1	482,000	432,000
9	Railroads, except those included with wharf structures and appurtenant drainage	675,000	639,000
10	Utilities to Wharf Area	102,000	82,000
11	Fencing, Site Grading & Drainage, and Miscellaneous	500,000	500,000
	Total	$ 27,971,825	$ 51,193,075
	+15% Engineering and Contingencies	4,195,775	7,678,960
	TOTAL ESTIMATED COST	$ 32,167,600	$ 58,872,035
	No. of Berths	15	20
	Cost Per Berth	$ 2,144,505	$ 2,943,600
	No. of covered berths	10	19
	No. of open wharves	5	1
	Gross Shed Area (sq. ft.)	1,000,000	1,900,000
	Area Behind Harbor Line Lost to Dredging (Ac)	0	106
	Cost Per Berth, Less Sheds (Incl. Engineering and Contingencies)	$ 1,876,175	$ 2,561,225

a) Cost comparison favors marginal wharves. This scheme has the least cost per berth (less sheds), $1,876,175 as compared to $2,561,225, and the least total cost, $32,167,600, as compared to $58,872,035 for the finger pier type.

b) The pier type layout can be utilized for the site, but is less suitable for berthing operations, in that the limited waterfront area provided by the relatively narrow ship channel is not as satisfactory as large open bays and wide rivers that usually prevail in the areas where finger piers are used.

c) The pier type layout provides more berths (20) than the other scheme, by utilizing land depth as well as channel frontage for berthing spaces.

d) The use of the pier type layout would result in the greatest amount of land (106 acres) lost to dredging behind the harbor line as compared to virtually none for the marginal wharf type.

e) The least cost for maintenance dredging would be for the marginal wharf type layout since the maintenance dredging of the ship channel (to within 50 feet of the fender line) is paid for by the Federal Government and the maintenance dredging of slips behind the harbor line would be paid for by the Navigation District.

f) The marginal wharf type requires a relatively small quantity (834,000 cubic yards) of dredged material from in front of the fender line (as compared to 13,800,000 cubic yards for the pier type), to be spoiled, thus selection of this layout would conserve the port owned dumping grounds which are needed for long term maintenance dredging operations in the turning basin area.

g) The pier and slip layout provides the best arrangement of facilities for railroad operations. The marginal wharf layout provides the best arrangement of facilities for trucking, steamship and stevedoring operations. Truckers, which now handle a major portion of the cargo, would have access to the full length and both ends of sheds in the marginal wharf layout.

h) The unbroken waterfront apron of the marginal wharves provides ready means of moving cargo along the waterfront in emergency operations from one shed to another shed or to a waiting ship up or down the channel.

i) In case of fire in a moored ship, one moored at a marginal wharf is more accessible to a fire boat than one moored in a slip. The U-shaped transit sheds of the pier type layout, with three sheds under one roof, present a greater hazard from fires and explosions than the marginal type.

RECOMMENDATIONS

Based on the findings and conclusions listed herein, it is recommended that the marginal wharf layout shown on Figure 4, estimated to cost $32,167,600, be adopted by the Port of Houston for the ultimate development of this area. Each of the schemes has advantages and disadvantages; however, the advantages of this scheme, which outweigh advantages of the finger pier type and which effected this recommendation, are briefly summarized as follows:

a) It is least in construction cost;

b) It provides the easiest berthing for ships, with least interference to other traffic in the channel;

c) It best satisfies the operational needs of steamship, stevedoring, and trucking companies;

d) It utilizes the land area to the greatest possible extent, with less land lost to dredging operations;

e) The cost of long term maintenance dredging to maintain project depth is least;

f) It best conserves the port owned dumping grounds for long term maintenance dredging operations;

g) The hazard from waterfront fires and explosions is least;

h) The unbroken wharf apron provides ready means for emergency handling of cargo between adjacent sheds or ships along the waterfront.

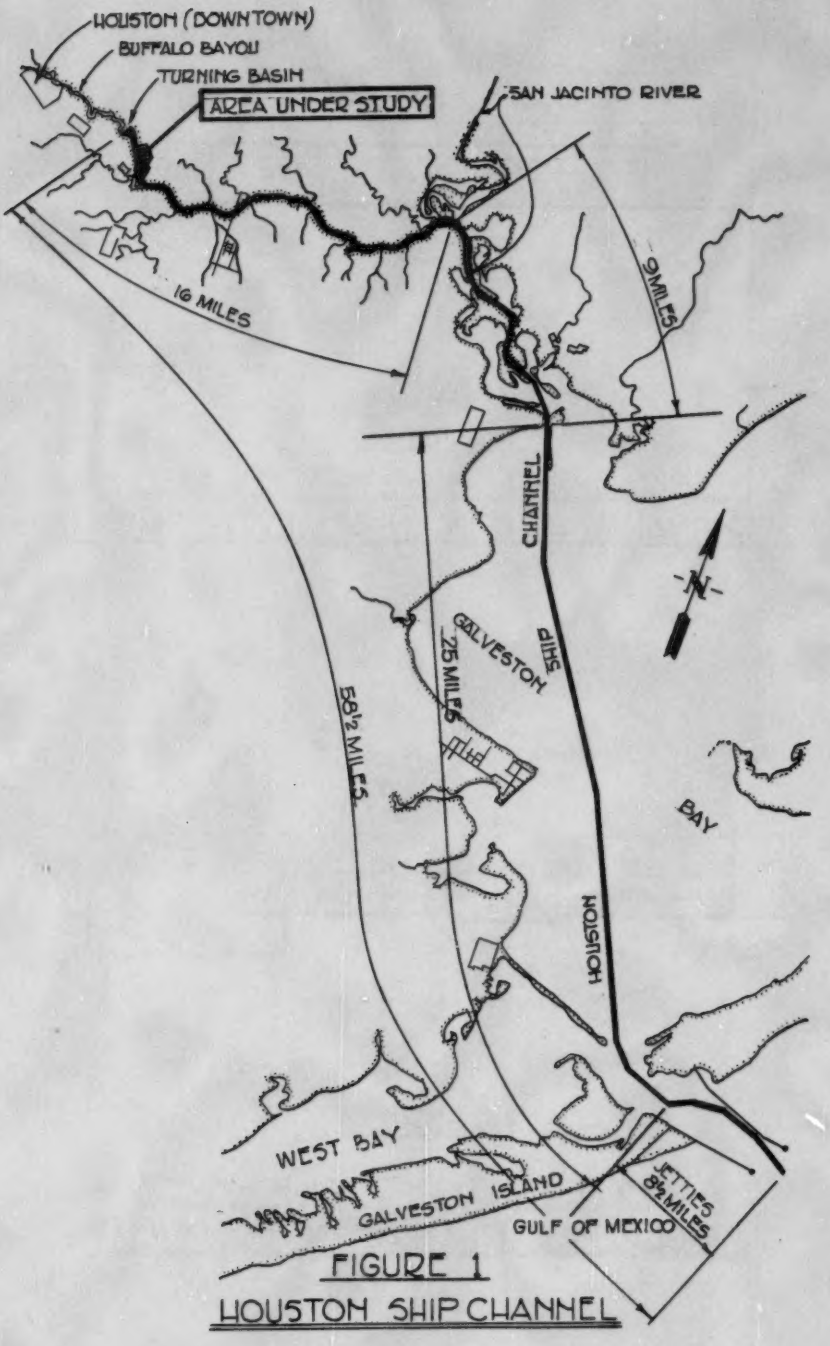


FIGURE 1

HOUSTON SHIP CHANNEL

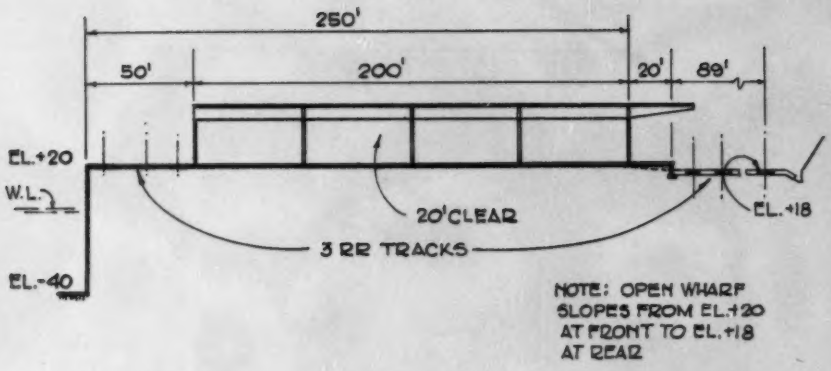


FIGURE 2

TYPICAL SECTION OF WHARF & SHED

MARGINAL WHARF PLAN OF DEVELOPMENT

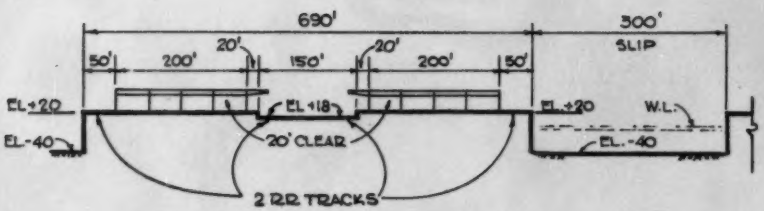


FIGURE 3

TYPICAL SECTION OF SLIP & SHED

FINGER PIER PLAN OF DEVELOPMENT

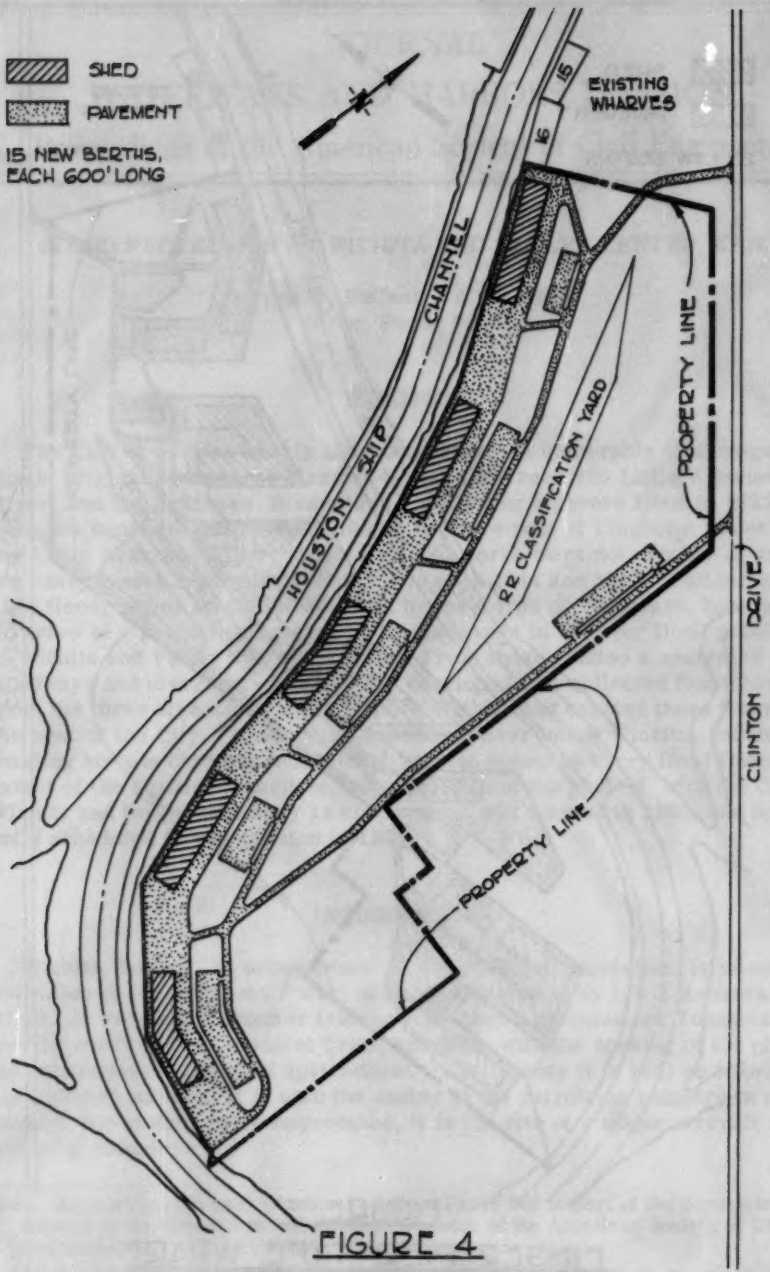


FIGURE 4
MARGINAL TYPE WHARF
PLAN OF DEVELOPMENT

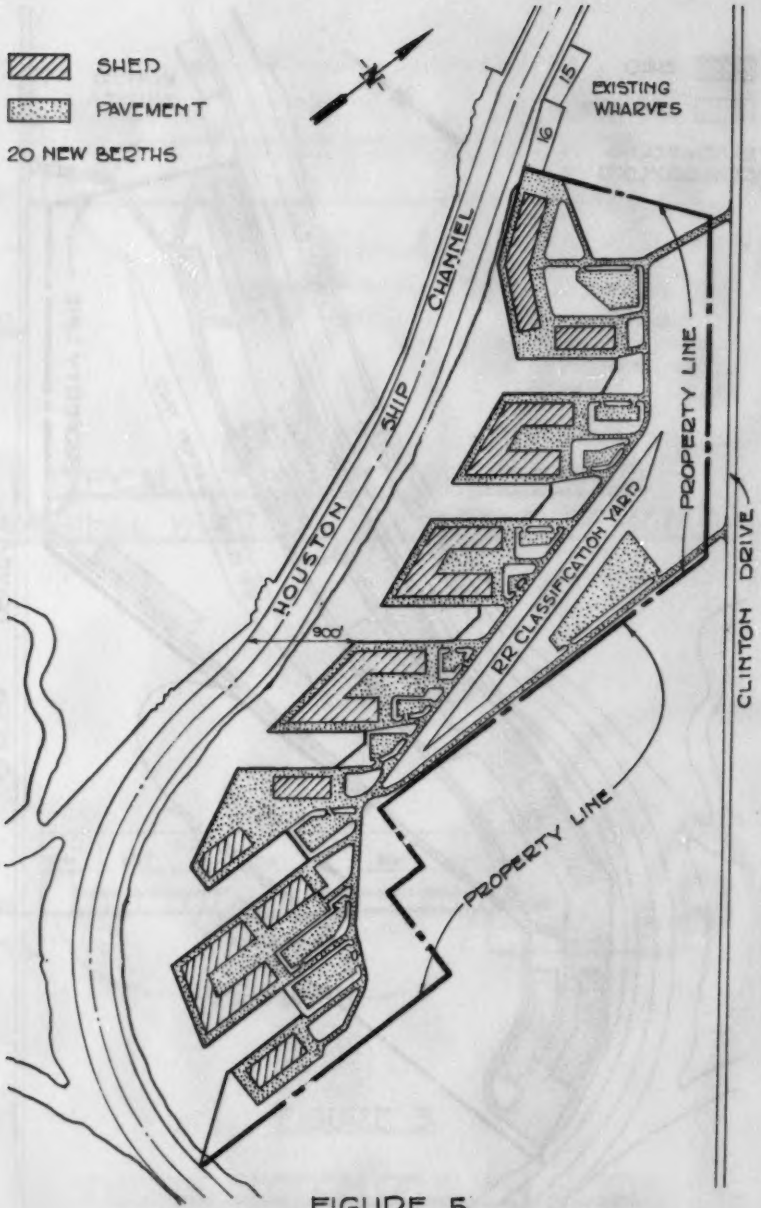


FIGURE 5

FINGER PIER TYPE WHARF

PLAN OF DEVELOPMENT

Paper 966 WW 3

JOURNAL
WATERWAYS AND HARBORS DIVISION
Proceedings of the American Society of Civil Engineers

FLOOD PROTECTION AT WICHITA AND VALLEY CENTER, KAN.

Myron W. DeGeer,[1] A.M. ASCE
(Proc. Paper 966)

SYNOPSIS

The City of Wichita and its suburban areas are vulnerable to damage from floods originating on three streams—Chisholm Creek, the Little Arkansas River, and the Arkansas River itself. Following a severe flood in 1923, local agencies expended substantial sums in improvement of Chisholm Creek and the Little Arkansas River. The completed works were not capable of preventing heavy losses from floods that occurred in 1944 and 1945. Following the 1944 flood, design studies were made by the Corps of Engineers, basically as a review of a project authorized by the Congress in 1936 for flood protection of Wichita and Valley Center, Kansas. From those studies a system of leveed floodways and diversion channels was developed that collected flood flows from the three streams at a point above Wichita and carried those flows to the west of the City, rejoining the Arkansas River below Wichita, and utilizing existing stream channels below the diversion points to carry flood flows to the extent of the channel capacities. Construction of the project, with the City of Wichita and Sedgwick County as cosponsors, was started in 1950, and is presently scheduled for completion in 1958.

INTRODUCTION

Wichita, Kansas, an urban center of over 200,000 population, is situated in the valley of the Arkansas River, at its junction with the Little Arkansas River. It was first a frontier trading post, then a railhead for Texas cattle moving north on the Chisholm Trail, and then, with the opening of the plains for settlement, a center of agricultural trade. Today it is still an important agricultural market. It is also the center of the petroleum industry in central Kansas, but of even more importance, it is the site of a major aircraft manufacturing industry.

Note: Discussion open until October 1, 1956. Paper 966 is part of the copyrighted Journal of the Waterways and Harbors Division of the American Society of Civil Engineers, Vol. 82, No. WW 3, May, 1956.

1. Asst. Chief, Eng. Div., Tulsa Dist., Corps of Engrs., U. S. Dept. of the Army, Tulsa, Okla.

The level terrain of the broad valley offers a pleasant site for a city. It simplifies many of the problems that often confront the builder in areas of greater relief. At Wichita, the first settlement was near the mouth of the Little Arkansas River, and through the years the City grew from that center, spreading north along the Little Arkansas and eastward across the Chisholm Creek drainage, without much attention being given to the possibility of flooding from these streams.

The urban area, however, was vulnerable to damage from three sources: Chisholm Creek, with its fan-shaped watershed lying in the low hills bordering the valley to the east and north; the Little Arkansas River, with its larger drainage in the rolling plains to the north and northwest, and reaching to the Kansas River Basin; and the Arkansas River itself, with its much larger basin to the west, rising in the Rocky Mountains, and near Wichita, receiving the inflow of the Pawnee River, Walnut and Cow Creeks, and other smaller tributaries.

By the 1920's, Wichita had grown to a population of nearly 75,000. Floods had not particularly hindered the City's growth, although there had been a major flood in 1877, which caused little damage because the developed urban area was small, and a flood of about the same magnitude in 1904, which caused severe damage to the area then developed. In June 1923, however, heavy rains over the Little Arkansas River and Chisholm Creek, when the Arkansas River was about three-fourths full, caused heavy damage in the City. The maximum flood limits on the Arkansas River, Big Slough and Cowskin Creeks, as shown on the map, Figure 1, were established by this flood.

During the next decade, over $1,000,000 was spent by the City, Sedgwick County, and the North Wichita Drainage District for channel improvement on Chisholm Creek and the Little Arkansas River. This work resulted in a system of channels and low levees that could control moderate floods in the better improved reaches, and could control minor floods throughout the urban area. Dredging of sand and gravel for construction material from the bed of the Arkansas River had been controlled by state agencies through the years, so that the Arkansas River channel through the City could carry the maximum floods of record.

Project Authorization

In the late 1920's, Congress authorized the Corps of Engineers to study most of the major river basins in the United States. Flood control, power, and other water problems on the Arkansas River were investigated under this authority. Studies of the Arkansas River at Wichita were included in the "308" report, as it was known.

Of the plans considered, that selected as preferable included draining the East Branch of Chisholm Creek into the Wichita Drainage Canal, rectifying and enlarging Chisholm Creek and diverting it into the Little Arkansas River, diverting the Little Arkansas River by a floodway beginning northwest of Valley Center and extending about three miles south to the Arkansas River, and constructing a floodway intercepting the Arkansas River at the mouth of the Little Arkansas River Floodway and following the general course of Big Slough to the Arkansas River downstream from Wichita, together with levees along the right bank of the Arkansas River from near Colwich to the Big Slough Floodway, and from the southern portion of Wichita to the lower end of the floodway. The estimated cost of these works was $2,895,615 for

construction and $1,774,560 for lands and damages, a total of $4,670,179.

Design flows for major elements of the project included Chisholm Creek Diversion, 11,000 c.f.s.; Little Arkansas Floodway, 25,000 c.f.s.; and Big Slough Floodway, 60,000 c.f.s.

The project selected in the "308" report as the most desirable was authorized for construction by the Congress in the 1936 Flood Control Act as "Wichita and Valley Center, on Arkansas River in Kansas and vicinity."

As with other projects authorized by that act, local interests were required to provide lands and rights-of-way; hold and save the United States free from damages due to the construction works; and maintain and operate all the works after completion.

For several reasons, including a public belief that the works then completed by the local agencies were adequate for protection from major floods, there was little interest in construction of the authorized project. Consequently, its initiation awaited an impetus that came only from disaster.

The 1944 and 1945 Floods

That disaster occurred in April of 1944. Rains occurring on April 22 and 23, averaging 4.87 inches over Chisholm Creek and 2.95 inches over the Little Arkansas, produced peak flows of 26,600 c.f.s. on the Arkansas River at Douglas Avenue; 26,300 c.f.s. in the Little Arkansas River at Valley Center; and 15,200 c.f.s. on the Wichita Drainage Canal at its upper end. Flooding lasted five days in Wichita, with the damage being heaviest in the industrial and stockyards area in north Wichita along Chisholm Creek and in the residential areas along the Little Arkansas River and the Wichita Drainage Canal. Maximum flood plain limits on Chisholm Creek were set by this flood. Estimated damage in the urban area was over $3,000,000.

This major flood was followed a year later by another of nearly the same magnitude. The April 1945 flood, however, was more severe on the Little Arkansas than the 1944 flood, establishing the flood plain limits shown on the accompanying map. Rains on April 10 through 16 of 4.48 inches over the drainage area produced a peak flow of 32,000 c.f.s. at Valley Center on April 16, with a peak flow of 19,700 c.f.s. on the Arkansas River at Wichita on April 17. There was only minor flooding on Chisholm Creek, as the heavy rainfall was concentrated in the Little Arkansas Basin. Damage from the flood was largely on the Little Arkansas, with total damage in the area being nearly $1,000,000.

Following the 1944 flood, there was a strong public demand for an organized effort to develop a plan for the control of floods in the urban area. As a result, civic leaders asked the Tulsa District, Corps of Engineers, to review the authorized project and, if possible, develop a plan for the protection of the area from future damage by floods.

In late 1944, studies were begun by the District, culminating in August 1945 in the completion of a definite project report.

The Design Problem

Briefly, the objective of the studies was to review the design of the authorized project, considering such elements as the urban development that had taken place in the area and the experience gained in design of such structures since the basic project studies were made. From this review, augmented by

additional studies of the hydrology of the area and with much more detailed survey and other basic data than were previously available, the authorized project design could then be retained or modified as required to provide the area with needed protection from floods.

The first step was development of design floods for the project. The storm of August 15-17, 1932, with center at Enid, Oklahoma, was selected for transposition as a basis for trial analyses, to develop comparable hydrographs from the various contributing areas, which could be modified by comparison with experienced data to obtain the project design floods.

As adopted, the project design flood discharges were based on a comparison with experienced floods over a period of more than 40 years and consideration of engineering and economic limitations. With the exception of East Branch of Chisholm Creek, the design discharges ranged from 69 to 100 percent greater than the maximum discharges of record. On the East Branch, the design discharge, which is 46 percent larger than the computed maximum experienced flood, was based on the capacity of the existing Wichita Drainage Canal, after improvement.

The maximum flood of record and the project design floods at key points are shown in Table 1.

Alternative Solutions

Two alternative solutions were found to be possible. Preliminary studies indicated that control of Chisholm Creek floods could not be economically accomplished by improving and enlarging the Wichita Drainage Canal, nor could these floods be diverted into the existing Arkansas or Little Arkansas River channels during concurrent floods on those streams. Therefore, it was concluded that either the channels of these two streams would have to be improved to carry flood flows from Chisholm Creek in addition to their own discharge, or large portions of the combined flows would have to be diverted through floodways.

Of these two basic plans, cost estimates for the plan contemplating improvement of existing channels indicated relatively high total costs for relocations and required internal drainage facilities, and high unit cost for right-of-way. The plan utilizing diversion through floodways had lower total costs for relocations, internal drainage facilities and rights-of-way; however, the construction cost was somewhat higher for this plan, so that the total cost for each of these basic plans was essentially the same. Both were feasible of construction and were economically justified.

Both plans were presented to the local interests for their consideration. Because of the difficulties involved in interior drainage with the river channel improved through the City and the indicated high cost of required relocations, and since it would not provide protection to as large an area as would the alternative floodway plan, the plan for diversion through floodways was adopted by the City of Wichita, the sponsoring local agency. This plan is shown on the accompanying map.

Essentially, this plan is identical with the authorized project. It was found during the studies that minor changes in the Chisholm Creek portion would result in a more economical facility, and the routing of the downstream part of the Big Slough Floodway was changed to also intercept Cowskin Creek and empty into the Arkansas River some miles below the mouth of Big Slough. This latter modification included part of another authorized project for flood

TABLE 1

FLOOD FLOW AT KEY POINTS

Point	Maximum Flood of Record (c.f.s.)	Project Design Flood (c.f.s.)
Chisholm Creek		
Above Mile 11.5 (1)	4,800	9,500
West Branch	2,400	4,700
Middle Branch	2,500	5,000
Above Mile 8.5 (2)	11,700	20,000
East Branch	4,100	6,000
Little Arkansas River at Valley Center	32,000	59,000
Arkansas River		
Above mouth of Little Arkansas River	16,600	31,500
Below mouth of Little Arkansas River	26,600 (3)	72,000

Notes: (1) Head of Chisholm Creek Interception Canal.
(2) Head of Chisholm Creek Diversion.
(3) Flow would have been 40,000 c.f.s. under confined conditions.

protection on the right bank of the Arkansas River below Big Slough, and protected several thousand additional acres of agricultural lands. Further, it avoided an area along Big Slough where an existing oil field and suburban developments would have increased the cost of right-of-way.

The Adopted Project

As developed from these studies, control of flood flows on Chisholm Creek, on the Little Arkansas and Arkansas Rivers, and on Big Slough and Cowskin Creeks would be accomplished by the following major units of the project, as shown on the map and as described below:

1) Big Slough-Cowskin Floodway
2) Little Arkansas River Floodway
3) Chisholm Creek Diversion
4) Arkansas River Levees
5) East Branch Interception Canal and Wichita Drainage Canal
6) Riverside Levees

Beginning at the hill line, where Big Slough enters the Arkansas River Valley from the west, about 15 miles northwest of Wichita, Big Slough will be diverted to the Arkansas River by a channel, leveed on the downstream side. This levee will continue as the right bank Arkansas River Levee for 13.5 miles downstream to the Big Slough-Cowskin Floodway. The Little Arkansas River Floodway will empty into the Arkansas River about six miles above the entrance to the Big Slough-Cowskin Floodway. This structure will intercept the Little Arkansas about two miles northwest of Valley Center and will be about three miles in length. The east bank Arkansas River Levee will extend from the mouth of the Little Arkansas Floodway to the mouth of the Chisholm Creek Diversion.

Chisholm Creek will be diverted by a leveed channel about three miles long, beginning at the mouth of Middle Branch and extending southwest across the Little Arkansas to empty into the Arkansas River at the Big Slough-Cowskin Floodway. Above the point of diversion, the channels of Middle Branch, Chisholm Creek, and West Branch will be improved and leveed to the hill line. Backwater levees will extend about one mile up the Little Arkansas.

From its beginning, the Big Slough-Cowskin Floodway will carry flood flows to the west of the City. It will intercept Big Slough two miles west of Wichita and Cowskin Creek about five miles southwest of Wichita, and will empty into the Arkansas River near Derby, about ten miles south of Wichita.

Other construction will include the improvement of East Branch of Chisholm Creek and of the Wichita Drainage Canal and the construction of the Riverside Levee along the right bank of the Arkansas River from the south side of Wichita to the mouth of the Big Slough-Cowskin Floodway.

Ungated control structures will be provided on the Little Arkansas River at the Little Arkansas Floodway, and at the Chisholm Creek Diversion; on the Arkansas at the upstream end of the Big Slough-Cowskin Floodway; and on Big Slough and Cowskin Creek where these streams will leave the floodway. These structures will be so designed that normal stream flows will follow their present courses, but flows during floods will be limited to channel capacity or less.

Interior drainage from the leveed areas will reach the channels through gated drainage structures, with provisions for ponding during flood periods.

Levee grades for the project will be based on four feet of freeboard on those levees protecting urban areas, and three feet where agricultural areas will be protected, except that where excess excavation is available a four-foot freeboard will be provided.

Because of the well-developed rail and highway net in the area, extensive alteration and relocation of these facilities will be required. On the Big Slough-Cowskin Floodway alone, construction of six railroad and three major highway crossings will be involved. To decrease the required length of bridges, the floodway section will be changed at these locations, with the depressed central channel being widened and the levees brought closer together. Generally, low chord on the bridges will be at least two feet above design water surface.

Other railroad and major highway changes will be required in connection with the Little Arkansas River Floodway and the Chisholm Creek Diversion.

The estimated cost of the project at the time of the definite project studies was $9,853,000. Of this amount, the Federal cost was estimated as $6,911,000 and the non-Federal cost, $2,942,000. Annual charges were estimated as $435,300 and annual benefits $439,100, so that the benefit-cost ratio was essentially 1 to 1.

Project Construction

On June 14, 1945, the Tulsa District was advised that the City of Wichita was willing and able to furnish the necessary assurances of local cooperation for the selected project plan. Formal assurances were submitted by the City on August 14, 1945, and on May 6, 1947, Sedgwick County joined the City as a cosponsor for the project.

The initial appropriation for construction was made by the Congress for fiscal year 1947; however, because of legal difficulties, the City was unable to proceed with the procurement of right-of-way at that time. Actual construction was initiated on May 8, 1950, on the East Branch Interception Canal and the Wichita Drainage Canal. This work was followed by initiation of the Big Slough-Cowskin Floodway on June 9, 1950.

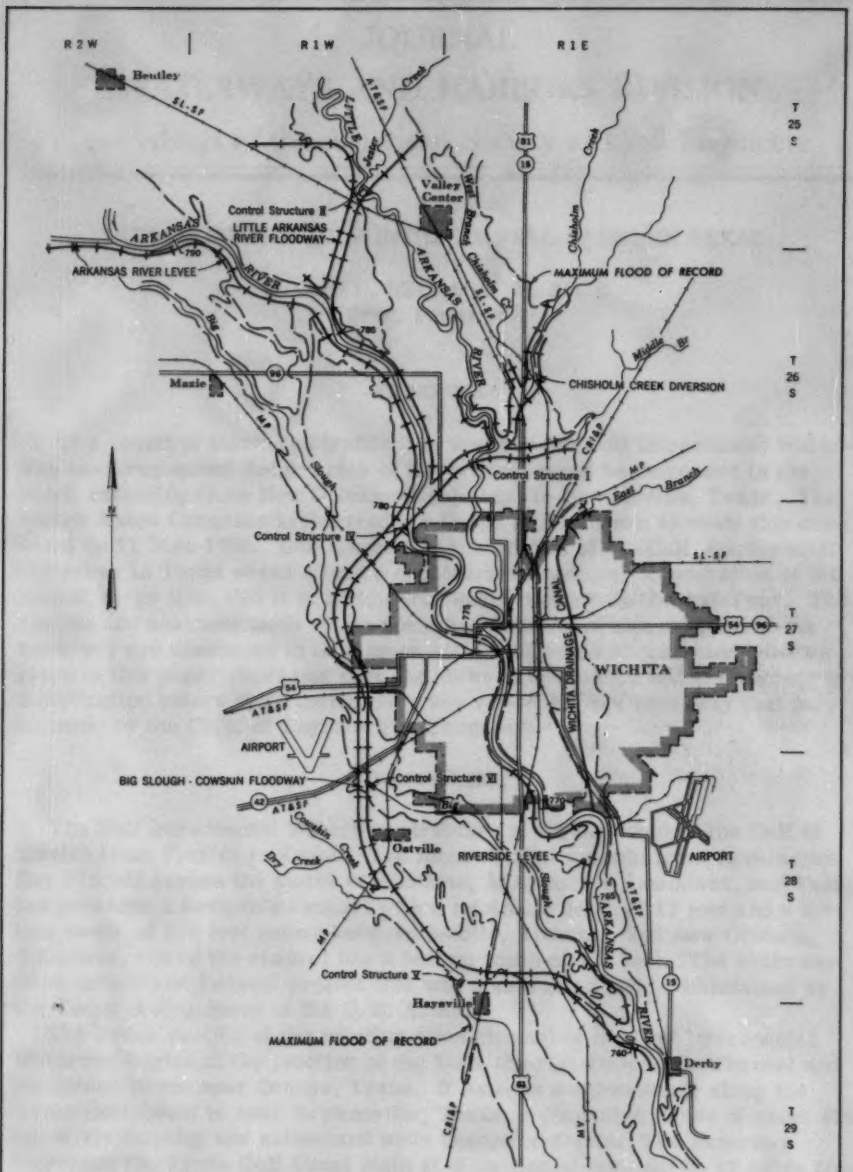
Construction of the Big Slough-Cowskin Floodway is complete except for railroad and highway relocations. It is anticipated that the floodway will be completed and that the diversion of the Arkansas River will be made by the late summer of 1956, thus providing protection from Arkansas River floods.

The Arkansas River Training Levees have been completed, and work will start on the Little Arkansas River Floodway in late 1955. Completion of this portion of the project is scheduled for the summer of 1957, followed by completion of the Chisholm Creek Diversion in the summer of 1958. The project is about 60 percent complete at this time.

Since 1945, costs of construction have increased materially, as have the costs of lands required for rights-of-way. As a consequence, the present estimated cost of the project is $19,580,000, of which $13,000,000 is Federal cost and $6,580,000 will be borne by the local interests; however, because of the continued growth of the City and the increase in value of the developed area, the project has a benefit-cost ratio greater than in 1945.

CONCLUSION

Losses from past floods at Wichita have been severe, and the potential damages from future floods are high. The flood problem can be solved by conventional means; however, that solution involves complex hydrologic and hydraulic design problems. Construction of the required floodways and diversion channels is expensive because of their magnitude, the high cost of right-of-way in this urban area, and the required alterations to the extensive rail and highway network. The construction of the needed works is under way, and will be completed in 1958.



WICHITA AND VALLEY CENTER
ARKANSAS RIVER, KANSAS
SCALE OF MILES

FIGURE 1

JOURNAL
WATERWAYS AND HARBORS DIVISION
Proceedings of the American Society of Civil Engineers

IMPROVING THE GULF INTRACOASTAL CANAL IN TEXAS

Willard P. McCrone,[1] M. ASCE
(Proc. Paper 967)

SYNOPSIS

As a result of increased traffic, the users of the Gulf Intracoastal Waterway have requested that the size of the present canal be increased in the reach extending from New Orleans, Louisiana, to Brownsville, Texas. The United States Congress authorized the Corps of Engineers to study this proposal on 11 June 1952. Enlargement of the section of the Gulf Intracoastal Waterway in Texas would afford a considerable economy in operation of the typical barge tows that it is anticipated would operate on the waterway. The studies and analyses made of the costs and benefits of enlargement of the waterway are discussed in this paper. The estimates of costs and benefits given in this paper represent only the views of the author and are subject to modification before their inclusion in any report on this waterway that may be made by the Chief of Engineers to Congress.

Description

The Gulf Intracoastal Waterway stretches along the coast of the Gulf of Mexico from Florida to Texas. The main channel extends from Appalachee Bay Florida across the states of Alabama, Mississippi, Louisiana, and Texas, and provides a navigable channel with a minimum depth of 12 feet and a bottom width of 125 feet except between Mobile, Alabama, and New Orleans, Louisiana, where the channel has a bottom width of 150 feet. The waterway is an authorized Federal project that was constructed and is maintained by the Corps of Engineers of the U. S. Army.

The Texas section of the existing main channel of the Gulf Intracoastal Waterway begins at the junction of the Lake Charles Deepwater Channel and the Sabine River near Orange, Texas. It extends southwesterly along the Texas Gulf Coast to near Brownsville, Texas, a channel distance of about 420 miles via existing and authorized main channel cut-offs. The waterway traverses the Texas Gulf Coast plain at distances of one-half to 22 miles from

Note: Discussion open until October 1, 1956. Paper 967 is part of the copyrighted Journal of the Waterways and Harbors Division of the American Society of Civil Engineers, Vol. 82, No. WW 3, May, 1956.

1. Colonel, Dist. Engr., Galveston Dist., Corps of Engineers, U. S. Dept. of the Army, Galveston, Tex.

the coast of the Gulf of Mexico, and lies, generally, in the low coastal marshes, a region of low relief crossed by river drainage systems and coastal bays. Adjacent land elevations range from mean low tide to three feet above. Depths in the lakes and bays crossed by the waterway range from less than one foot to 13 feet deep. Many branch channels in the bays and rivers have been improved and constitute "feeders," or tributary channels, to the main channel. Plate 1 shows the location of the Gulf Intracoastal Waterway and adjacent area in Texas.

From the eastern end of the Texas section, the main channel coincides with the deepwater channels of the Sabine-Neches Waterway that extend along the Sabine River and the northwest shoreline of Sabine Lake to near the southern limit of Port Arthur, Texas, a distance of about 23 miles. These deepwater channels have authorized dimensions of 30 feet deep and 200 feet wide in the Sabine River and across the northern part of Sabine Lake to the Neches River, a distance of about 11 miles, and 36 feet deep and 400 feet wide in the other 12 miles of deep channel.

From the southern limit of Port Arthur, the main channel extends southwesterly, generally across low lying marshy lands, to the deepwater Houston Ship Channel in Galveston Bay near Port Bolivar, a distance of about 62 miles. See photo 1. In the vicinity of Port Bolivar, the main channel connects with the deepwater Texas City Channel and has a connection with the deepwater Galveston Channel. From the Houston Ship Channel, the main channel extends in a southwesterly direction for a distance of about 44 miles to an intersection with the deepwater channel of Freeport Harbor, thence for about 140 miles to a connection with the deepwater Port Aransas-Corpus Christi Waterway near Port Aransas, Texas. See photos 2 and 3 of major river crossings in this section. In this 189-mile reach, about 43 miles of the main channel traverses lands varying from low salt marshes and tidal flats to marginal grazing lands with an elevation of about two feet. Seventy-two miles of the channel are located adjacent to the north shore of coastal bays where dredging spoil placed on the bayside of the channel has provided a protective embankment averaging about 5 feet high. Relocation of the main channel along the northwest shore of Redfish Bay between Aransas Bay and Corpus Christi Bay, a distance of about 17.2 miles, authorized by the River and Harbor Act of July 24, 1946, has not yet been undertaken because funds have not yet been appropriated for this purpose. About 79 miles of the channel are located across open coastal bay waters.

Going southwest, the main channel coincides with the Port Aransas-Corpus Christi Waterway and the deepwater Encinal Peninsula Channel for a distance of 12 miles extending from Port Aransas to a point about 12 miles east of Corpus Christi, Texas. Thence the channel extends southward across Corpus Christi Bay to Laguna Madre and through the Laguna Madre to the turning basin at Port Isabel, Texas, a distance of 126 miles. The main channel then extends, through the deepwater channels to Port Isabel and Brownsville, to the turning basin near Brownsville, a distance of about 15 miles.

The Texas section of the Gulf Intracoastal Waterway is tidal throughout its length. The mean tidal range is 0.5 foot at Orange, 1.0 foot at Port Arthur, 1.3 feet in Galveston Bay, 1.5 feet at Freeport, 1.0 foot in Matagorda Bay, 1.0 foot in San Antonio Bay, 1.0 at Corpus Christi, 1.5 feet at Port Isabel, and 1.0 foot at Brownsville. The height of tides is dependent, largely, on the wind blowing at any given time. Strong north winds have depressed the water surface as much as 2 feet while tropical hurricanes have caused heights as much as 15 feet above mean low tide.

There are nine shallow-draft tributary channels to the Texas section of the waterway, authorized under the project for the Gulf Intracoastal Waterway. There also are six shallow-draft channels, authorized under different projects, that are tributary to the Intracoastal Waterway in Texas. These channels, with project dimensions and stage of completion, are listed in table 1:

TABLE 1

AUTHORIZED TRIBUTARY CHANNELS TO GULF INTRACOASTAL WATERWAY IN TEXAS

Name of channel	Authorized dimensions: Depth below MLT (feet)	Bottom width (feet)	Turning basin (feet)	Length (miles)	Completed length (miles)
Tributaries under the Gulf Intracoastal Waterway project					
San Bernard River Channel	9	100	(1)	31.0	26.0
Colorado River Channel	9	100	400x500	17.0	0.0
Channel to Palacios	9	100	(1)	5.7	5.7
Channel to Rockport	9	200	(1)	2.0	2.0
Channel to Aransas Pass	12	125	300x2200	6.1	6.1
Channel to Barroom Bay	6	60	None	0.4	0.0
Channel to Victoria	9	100	(1)	45.5	13.0
Channel to Port Mansfield	12	125	400x600	1.4	1.4
Arroyo Colorado to Harlingen	12	125	400x500	33.0	25.7
Tributaries under separate Federal projects					
Trinity River, Channel to Liberty	9	150	(1) 500x500	48.9	23.2
Cedar Bayou, Texas	10	100	None	12.5	3.2
Clear Creek & Clear Lake	7	60	None	9.2	9.2
Dickinson Bayou	6	60	None	11.4	11.4
Chocolate Bayou	4	100	None	3.0	3.0
Port Lavaca Channel	9	100	None	42.2	22.0

(1) Turning basins are to be provided by the local interests.

History

The development of protected navigation channels along the Texas coast was one of the very earliest public and private improvements. As early as 1874, the State of Texas constructed a shallow-draft channel between Aransas Bay and Corpus Christi Bay. A number of other small channels were dredged by public and private interests through shell reefs in the coastal bays and through the land separating the bays. Federal improvement of these connecting shallow-draft navigation channels in Texas started about 1892, with the first early work on the route of the present Intracoastal Waterway. The channel at that time was 5 feet deep by 40 feet wide, which was ample for the shallow-draft sail boats and launches that navigated the coastal bays. In 1925, studies of the requirements of barges and towboats in use at that time led to the authorization of dimensions of 9 feet in depth and 100 feet in bottom width

for the main channel of the Gulf Intracoastal Waterway between Corpus Christi and the Sabine-Neches Waterway.

During the 1930's, the increase in size and speed of towboats and barges and the rapidly expanding commerce moved over the waterway taxed the capacity of the channel. On July 23, 1942, enlargement of the channel to a depth of 12 feet and a bottom width of 125 feet was authorized by the Congress of the United States throughout the length of the Intracoastal Waterway in Texas, from the Sabine River to the Rio Grande.

In the post-war years, the amount of commerce carried on the enlarged Gulf Intracoastal Waterway has multiplied many times. The barges and towboats increased in size and the towboats increased in power. The equipment now in use on the channel is unable to operate at full efficiency because of the restricted depth and width of the channel. Because of this condition, the Corps of Engineers was authorized by Congress to again study the feasibility and economic justification of further enlargement of the waterway.

Investigations

Investigation of navigating conditions revealed that large tows of 2 and 3 or more barges with widths up to 54 feet and loaded drafts of 9 to 10 feet were being operated on the channel at a low efficiency. The excessive drag on these tows at normal operating speeds required greatly increased power from the towboats, and, in most instances, reduced the speed of tows in the main channel by an appreciable amount. The object of the study was to determine the enlargement of the waterway that could be justified by the saving in transportation costs that would be realized by more efficient operation of tows on the waterway.

In order to determine the most feasible plan of channel enlargement, studies were made of channels of various widths and depths. Preliminary studies indicated that deepening of the channel would provide the cheapest method of obtaining more efficient vessel operation. Accordingly, detail studies were limited to channels with depth of 18 feet and widths on bottom of 125,200,250 and 300 feet. Estimates of costs of the channel enlargements were based on studies of earth excavation, rights-of-way and spoil area requirements, and pipe line, utility, and bridge alterations. Detailed studies were made of the cost of maintenance of the existing and enlarged channels.

One question that arose, early in the planning studies of this project, was the question of enlarging the numerous structures that cross the canal. These structures include bridges, locks, pipe lines, and aerial wire crossings. Pipe line and aerial wire crossings can be altered at relatively small cost; however, the bridges and locks present a much more costly problem.

There are 13 bridges across the main channel of the Gulf Intracoastal Waterway in Texas, of which two are railroad bridges and 11 are highway bridges. Only one of the bridges is a fixed structure. It has a horizontal clearance of 201 feet and a vertical clearance of 73 feet above mean high water. The movable bridges are of three types; bascule, swing, and ponton. These bridges have horizontal clearances varying from 100 feet to 200 feet and unlimited vertical clearances when opened. Flood gates, in the Gulf Intracoastal Waterway at the Brazos River crossing, have horizontal clearances of 75 feet and controlling depths on the gate sills of 15 feet at mean low tide. See photo 2. Navigation locks in the Gulf Intracoastal Waterway on each side of the Colorado River crossing have the same controlling dimensions as the

flood gates and have lock chambers 1,200 feet long. See photo 3. Since the through traffic must utilize the canal in Louisiana, the limiting dimensions there are pertinent to any modification proposed for Texas. In Louisiana, there are 11 bridges across the main channel with minimum horizontal clearance of 75 feet. All of these bridges have movable spans. There are four navigation locks across the waterway in Louisiana, with depths of 12 to 13 feet over the mitre sills and widths, between the lock walls, of 56 to 75 feet.

Three of the bridges and navigation structures were designed for a channel 9 feet deep by 100 feet wide and the remainder were designed for a depth of 12 feet and width of 125 feet. Operating regulations do not allow passing of tows in lock or bridge structures. Widths through structures are adequate for tows of single barge width. The cost to widen the gate structures and most of the major bridges to permit double barge tows, or passing in the structures, was quickly determined as prohibitive. The sills of the lock and gate structures now have limiting depths varying from 12 feet to 15 feet at mean low tide. The cost of lowering these sills would be very expensive, and, because of the desired use of towboats on feeder and other channels in the waterways connecting to the canal, it is very doubtful if users would adopt deeper draft towboats for Intracoastal Waterway use. In addition, speed is normally greatly reduced in passing through the gates and bridges because of the hazards of collision, with the result that operating efficiency would be low regardless of the available depth or width. For these reasons, it was decided that no enlargement of the channel would be provided within 1,000 feet of the center line of the bascule and swing bridges or of the end of the guide walls at locks and gates.

The one large fixed bridge at Freeport (Surfside) has an opening of 201 feet and the channel can be deepened to 18 feet with no modification of the bridge. The ponton bridges are of relatively inexpensive construction consisting of two shore trestles and a swing barge. The cost to lengthen the barge and relocate one trestle would be minor. It is proposed that the deepening and widening be carried out through all ponton bridges. The proposed deepening would be carried through the overhead bridge at Freeport, but no widening would be proposed through this structure.

The reaches through bascule and swing bridges and locks, which would not be improved, have a total length of 7.1 miles. The overall length of the main channel of 418.0 miles includes the length of channel that coincides with the Sabine-Neches Waterway from Orange to Port Arthur, with crossings of the Houston Ship Channel, the Freeport Harbor, and the Port Aransas-Corpus Christi Waterway and with the Brazos Island Harbor project from Port Isabel to Brownsville. The deep-water sections have a total length of 38.0 miles. These deep-water sections are already larger than the proposed improvements in this study, so no work will be required for these sections. A total of 45.1 miles of channel would not be improved.

Modification of Channel Alinement

One of the first considerations, in planning a major enlargement of the channel of the Gulf Intracoastal Waterway, is improvement of the channel alinement. A study of the existing alinement showed that curve easing of a number of curves and several major relocations would be desirable.

Proposed Curve Easing

In general, the alinement of most of the existing channel under consideration for enlargement, consists of comparatively long tangent courses. There are a total of 113 curves in the 381 miles of channel between the Port Arthur Canal and the Port Isabel turning basin. Of these, 18 curves are between Port Arthur and the Houston Ship Channel, a distance of about 62 miles; 80 curves are between the Houston Ship Channel and the Port Aransas-Corpus Christi Waterway, a distance of about 189 miles; and, 15 curves are between the Port Aransas-Corpus Christi Waterway and Port Isabel, a distance of about 128 miles. The maximum curving of the channel alinement is 2 degrees 18 minutes, which corresponds to a radius of 2,491 feet.

Traffic on the Gulf Intracoastal Waterway consists of vessels of all types from small pleasure boats to integrated tows of 2 or more barges. The multiple barge tows approximate the maximum size permitted by existing regulations, which limit the length to not more than 1,000 feet, exclusive of the towboat, and the width to not more than 55 feet. Passage of maximum length tows through present curves of maximum curving is hazardous and unsafe to navigation.

Investigation of the feasibility of easing the curves on the waterway to a maximum of one degree, or about 5,730-foot radius, revealed that such easing could be accomplished without requiring excessive rights-of-way and with a comparatively small amount of dredging. Such curve easing is more than justified to provide for reasonably free, easy and unobstructed navigation through the curves of the waterway. Accordingly, all plans of improvement provide for easing of curves to provide a maximum of one degree. Excluding the curves located within the limits of proposed channel relocations, a total of 17 curves would be eased and 9 curves would be eliminated under the proposed plan.

Channel Relocations

Channel relocations were studied in the following reaches of the waterway:

a) East approach to GC&SF RR bridge - Mile 319.7
b) Curves west of GC&SF RR bridge - Mile 320.7 to 326.0
c) Entrance to Matagorda Bay - Mile 458 to 475
d) Across Corpus Christi Bay
e) Near Port Isabel

Relocation of the channel at the east approach to the GC&SF RR bridge would relieve a hazardous reach of channel that causes considerable loss of time for barge tows because of the slow speed and delays experienced in the existing channel. The relocation would increase the channel length about 0.27 mile, but would reduce the hazards to navigation in the reach, particularly the danger of collision with the bridge. The first cost of this relocation is estimated at $220,000 and the annual charges are estimated at $8,000. See photo 1.

Just west of the GC&SF RR bridge is a 5.3 mile reach of channel that contains 9 curves, with a maximum degree of 2 minutes and 18 seconds. Rectification of this reach would reduce the length of channel by 0.67 mile, and would reduce the delays and hazards to navigation involved in traversing the existing route. The relocation would have a first cost estimated at $386,000 and annual charges estimated at $12,800.

The relocation at the entrance to Matagorda Bay would return the channel to a former location. During the Second World War, military operations in Matagorda Bay endangered navigation on a portion of the channel in the northeast part of Matagorda Bay. To avoid this danger, the channel was relocated on its present route. Experience has shown that maintenance of the original route would be considerably less than maintenance of the present route. The original route is about 0.04 mile shorter than the existing route and would be a safer route for barge traffic because of the greater length of land cut and better alignment with prevailing winds. The first cost of redredging the original route is estimated at $374,000 and the net annual charges, with credit for the saving in annual maintenance, are estimated at $700.

The relocation in Corpus Christi Bay would be an extension of the authorized alternate route near Aransas Pass straight across the bay to the channel in the south part of Corpus Christi Bay. Connecting curves would be provided at the junction with the deep-water Port Aransas-Corpus Christi Waterway. The relocation would shorten the travel distance for all through traffic on this reach by 1.8 miles, would reduce the number of curves in the channel, and would eliminate the hazards to navigation encountered by barge traffic from the Gulf Intracoastal Waterway traversing the deep-draft Port Aransas-Corpus Christi Waterway. The first cost of this relocation is estimated at $82,000, and the annual charges are estimated at $4,600.

The relocation at Port Isable would improve the approaches to a bridge that crosses the channel in the Laguna Madre. The realined channel would be about 0.6 mile longer than the present route. The relocation would ease the curves to a 5,730-foot radius and would materially increase the safety and convenience of navigation. The first cost of this relocation is estimated at $200,000 and the annual charges are estimated at $10,000.

The channel relocations and rectifications would reduce the length of the main channel in Texas by about 1.6 miles. The total length of the main channel through the channel relocations would be 416.4 miles. The length of channel in land cuts, in water cuts, in the reaches that would not be improved, and the total length and the length of channel to be improved are summarized for the three principal reaches of the waterway in Texas, in table 2:

TABLE 2

LENGTH OF CHANNEL

Section of waterway	Channel length (miles) (1)				
	To be improved			Not to be improved	Total
	Land	Water	Total		
Sabine River to Galveston Bay	56.5	3.6	60.1	24.1	84.2
Galveston Bay to Corpus Christi Bay	104.6	78.6	183.2	6.2	189.4
Corpus Christi Bay to Brownsville	1.2	126.8	128.0	14.8	142.8
Total	162.3	209.0	371.3	45.1	416.4

(1) Length is via the authorized but not constructed alternate route at Aransas Pass.

The land cuts traverse prairie or marsh lands ranging in elevation from one to three feet above mean low tide, on which disposal of material dredged from the existing channel has created a more or less continuous spoil embankment along the channel. The spoil bank has an average height of about 5 to 6 feet and a maximum elevation of 10 to 12 feet. In planning the enlargements, the necessary widening of the channel is proposed, wherever possible, on the side opposite the spoil banks. In many reaches, however, existing improvements and the high cost of rights-of-way dictate widening on the same side as the spoil bank, which involves removal of portions of the spoil bank.

The 209.0 miles of channel in the open bays includes a number of reaches in Matagorda Bay, Aransas Bay and Corpus Christi Bay that are in depths of over 12 feet. These reaches do not require dredging under the present authorized depths of 12 feet. However, deepening the project depth to 18 feet would require dredging along the entire length of channel. In the water sections, spoil disposal is accomplished along the channel at distances of about 600 feet from the channel. The required widening of these reaches is proposed generally along the same side as the spoil disposal areas in order to avoid interference with dredging operations by passing traffic as much as possible. Disposal on the opposite side of the channel would result in the interference with dredging each time a vessel or barge tow passed in the canal.

Channel Excavation

The proposed channel improvements are enlargements of existing or authorized project channels. The existing channel was constructed and subsequently maintained by hydraulic pipeline dredges. The materials to be encountered in dredging the proposed enlargements, based on previous dredging borings and on borings in undredged channels, consist of silts, clays, sands, and shells of various thicknesses and mixtures. Between miles 580 and 595 in the Baffin Bay area, there are exposures of thin layers of sandstone formations. Similar formations, encountered while dredging the existing channel, were removed by under cutting and without blasting. The materials to be excavated for the channel enlargements should offer no unusual difficulties for hydraulic dredging. The estimates of unit dredging costs are based on the use of modern 32-inch suction and 27-inch discharge pipeline dredges. Spoil-disposal areas are available in close proximity to the dredging work. It is estimated that the output of the dredge would probably average 750,000 cubic yards per month. The quantity of excavation that would be required for the several enlargements of the channel section are given in table 3.

Rights-of-way

Laws for Federal participation in the improvement of navigable waterways require the local interests to furnish all necessary lands, rights-of-way, and spoil-disposal areas for initial construction and for subsequent maintenance. This includes the cost of removal of all structures on the right-of-way, and alteration of pipe line and utility crossings. In the open water sections of the Gulf Intracoastal Waterway no rights-of-way are required, since the United States has a paramount right to use submerged lands under navigable waters for improvements for navigation. The land cuts, however, require considerable right-of-way for construction of the channel enlargement and for disposal of spoil materials. The channel right-of-way required includes the top

TABLE 3

ESTIMATED QUANTITIES OF EXCAVATION

Size of channel (feet)	Section of waterway and quantity of excavation in thousands of cubic yards			
	Port Arthur to Houston Ship Channel	Houston Ship Channel to Port Aransas-Corpus Christi Waterway	Port Aransas-Corpus Christi Waterway to Port Isabel	Total in Texas
18 x 125	17,651	49,203	31,437	98,291
18 x 200	37,586	98,476	66,244	202,306
18 x 250	50,753	136,120	88,188	275,061
18 x 300	54,511	175,904	110,148	340,563

of the cut plus a berm of 100 feet on each side of the channel edge. The berm is for work area and for possible erosion. The increase in channel right-of-way varies from 157 feet for an 18' x 125' channel to 332 feet for an 18' x 300' channel, assuming an average ground elevation of 4 feet.

In general, the present spoil-disposal areas could be used for deposition of spoil from the proposed enlargement. These spoil areas would have to be enlarged considerably for the maximum channel enlargement under study, and would then extend for about 1,200 feet from the edge of the channel right-of-way.

In disposing dredging spoils in the open bay close coordination is maintained with the State Game and Fish Commission and the United States Fish and Wildlife Service. The spoils are deposited, as closely as can be economically justified, in accordance with the ideas of those authorities to prevent damage to the natural fish and wildlife resources in the bays. This, generally, is done by staggering the spoil banks on opposite sides of the channel to permit circulation of water in the bays across the channel.

Estimates of Cost

First Cost

The unit prices of construction used in estimating the first cost of the proposed channel enlargements are based on the costs of similar work performed in the Galveston District during August 1955. The estimated unit prices for dredging range from about 10 cents a cubic yard to about 20 cents a cubic yard depending on the quantity of excavation, the character of materials to be excavated, and the accessibility of the site of the work. The prices of land for rights-of-way are based on a study of land values and recent land sales in the vicinity of the canal. The cost and requirement of aids to navigation were obtained from the U. S. Coast Guard, which is charged with the responsibility of installation of navigation aids in Federal projects. Costs of other work involved in the improvement were estimated on the basis of unit prices of similar work in the vicinity of the waterway. The estimated first costs of the several improvements are summarized in table 4. The costs given in table 4

include the costs of relocations, as well as the cost of deepening and widening the channel.

TABLE 4

ESTIMATES OF FIRST COST

Size of channel (feet)	Section of waterway and estimated first cost			
	Port Arthur to Houston Ship Channel	Houston Ship Channel to Port Aransas-Corpus Christi Waterway	Port Aransas-Corpus Christi Waterway to Port Isabel	Total in Texas
18 x 125	$3,071,000	$7,540,000	$8,102,000	$18,713,000
18 x 200	6,439,000	14,396,000	16,518,000	37,353,000
18 x 250	8,542,000	19,121,000	21,643,000	49,306,000
18 x 300	10,848,000	24,159,000	26,827,000	61,834,000

Annual Charges

The estimates of annual charges include interest and amortization on the investment in the improvement plus the increase in annual maintenance cost of the project resulting from the enlargement of the channel. The investment includes interest during the construction period, and the amortization is based on a 50-year useful life of the improved channel. Navigation aids are considered to have a 25-year useful life. The estimates are based on a rate of interest of 2-1/2 percent for all public costs in the improvement and 4 percent for private costs.

The additional cost to adequately maintain the enlarged channel would be largely proportional to the additional bottom width of channel to be maintained. The increase in depth is considered to have little influence on the rate of shoaling in the channel. Factors considered in estimating the maintenance of the enlarged channels are the maintenance of the existing channel, the location of the channel in land cut or in open water, and the increase in width of the channel. The estimated annual charges for the several sizes of channel enlargement are given by reaches in table 5.

TABLE 5

ESTIMATES OF ANNUAL CHARGES

Size of channel (feet)	Section of waterway and estimated annual charges			
	Port Arthur to Houston Ship Channel	Houston Ship Channel to Port Aransas-Corpus Christi Waterway	Port Aransas-Corpus Christi Waterway to Port Isabel	Total in Texas
18 x 125	$114,000	$ 285,000	$ 303,000	$ 702,000
18 x 200	264,000	809,000	766,000	1,839,000
18 x 250	356,000	1,251,000	1,068,000	2,675,000
18 x 300	455,000	1,694,000	1,371,000	3,520,000

Benefits

Channel Relocations

The benefits of the five major channel relocations were estimated separately from the benefits of the channel enlargement. These benefits include the saving in time because of shorter travel distance, an estimate of the saving in prevention of collisions, and the value of the increased safety and convenience to navigation. The estimated benefits for each relocation are shown in table 6, which also shows the annual cost of the relocation of the 12 x 125-foot channel and the ratio of annual benefits to annual charges.

TABLE 6

COMPARISON OF BENEFITS AND COSTS
MAJOR RELOCATIONS OF CHANNEL

Site of relocation	Annual benefits	Annual charges	Ratio
East approach to GC&SF RR bridge	$10,000	$ 8,000	1.3
Just west of GC&SF RR bridge	56,600	12,800	4.4
Entrance to Matagorda Bay	5,200	700	7.4
Across Corpus Christi Bay	10,830	4,600	2.4
Near Port Isabel	2,000	10,000	0.2

Channel Enlargement

The benefits from enlargement of the main channel of the Gulf Intracoastal Waterway would result from a saving in the cost of transportation through more efficient operation of towing equipment on the enlarged channel. The saving is evaluated as the saving in ton-mile cost of moving the commerce over the enlarged waterway instead of over the existing waterway.

The traffic in ton-miles used in this analysis, is based on the commercial statistics compiled by the Corps of Engineers for the year 1953, which were the latest detailed figures available when the analysis was made. The ton-miles of traffic that moved over only the reaches of the Gulf Intracoastal Waterway that would be enlarged are given in the following tabulation:

Section of waterway	Ton-miles of traffic (1953)	
Port Arthur to Houston Ship Channel	774,500,000	
Houston Ship Channel to Corpus Christi Bay	976,600,000	(1)
Corpus Christi Bay to Port Isabel	54,400,000	

In order to evaluate the saving in operating costs of the barge traffic on the enlarged waterway, it was considered sufficient to analyze the operation of two typical tows. Obviously there is a large variety of towboats and barges, so the selection of "typical" types is difficult. The sizes selected are based on the newest equipment in use and the sizes currently under construction. It is assumed that as the older equipment wears out, it will be replaced with

(1) Includes 3,400,000 ton-miles through the Freeport bridge which would receive benefit in part of the plans for widening and deepening the channel.

approximately the typical sizes selected. The tows selected as typical are as follows:

a) For through traffic - 3 - 50' x 250' barges with a 1600 horsepower towboat.
b) For local traffic - 2 - 35' x 195' barges with an 800 horsepower towboat.

The hourly operating costs of these typical tows are estimated from data given in the Waterways Economic Series No. 5 prepared by the Board of Engineers for Rivers and Harbors. The costs in this bulletin are based on 1951 prices. The hourly cost and other characteristics of the two tows are as follows:

Item	Typical tow	
	Through traffic	Local traffic
Towboat, horsepower	1,600	800
Barges, size	3 - 50' x 250'	2 - 35' x 195'
Displacement, tons	9,250	3,340
Cargo, tons	7,680	2,770
Cost of towboat an hour	$44.59	$27.29
Cost of barges an hour	$11.10	$ 3.68
Cost of tow an hour	$55.69	$30.97
Estimated cost, 1955 prices	$64.04	$35.62

The characteristics of operating these two typical tows over the several sizes of channel were determined from data prepared by the University of Michigan[1] from model tests, showing the relation of barge-tow speed to resistance encountered in moving over channels of different sizes, and from data prepared by Dravo Corporation[2] from model tests of the relation of towboat speed to the push or thrust of the towboat for overcoming tow resistance.

The analysis is made by plotting the thrust and resistance against speed on the same graph. The intersection of the two curves gives the optimum speed attainable for the typical tow in a given size channel. The analysis is indicated by the curves for the two typical tows in channels 12 feet deep by 125 feet wide and 18 feet deep by 125 feet wide. These curves are shown on plate 2.

The thrust curve for the 1,600 horsepower towboat is interpolated between the thrust curves for 1,500 and 2,000 horsepower given by Dravo and the thrust curve for an 800 horsepower towboat is interpolated between the thrust curves for 500 and 1,000 horsepower towboats given by Dravo.

The resistance curves for the 2 - 35' x 195' barge tows are taken directly from the University of Michigan data. The resistance curves for the 3 - 50' x 250' barge tows are obtained by modifying the University of Michigan data for 2 - 54' x 250' barge tow. The resistance of a 3 - 50' x 250' barge tow was estimated by assuming that the resistance of the third barge would be 70 percent of the resistance of each barge in a two barge tow. This gives a

1. Experimental data prepared by the University of Michigan for the Pittsburgh District of the Corps of Engineers - completed in 1954.
2. Chart X in the Development and Economic Potential of Inland Waterway Transportation - Dravo Corp. - July 1949.

factor of 135.0 percent of the resistance of the two barge tow. A further factor of 92.5 percent, obtained by assuming that the unit resistance is proportional to the width of the barges, was applied to account for the narrower barges. The resultant factor is 1.25 percent of the total resistance of the 2 - 54' x 250' barge tow.

The savings in transportation costs obtained from applying these curves, as shown on plate 2, are given as follows:

Tow and size of channel	Speed in miles/hr	Cargo ton-miles/hr	Cost per hr(mills)	Cost per ton-mile (mills)
3-50'x250' barges, 1,600 hp				
12' x 125' channel	4.64	35,700	64,040	1.79
18' x 125' channel	6.03	46,300	64,040	1.38
Saving per ton-mile of cargo				0.41
2-35'x195' barge, 800 hp				
12' x 125' channel	5.67	15,700	35,620	2.27
18' x 125' channel	6.92	19,170	35,620	1.86
Saving per ton-mile of cargo				0.41

From this analysis it appears that the unit saving per ton-mile is the same for the large and small tows. As a result, it was not necessary to segregate the through and local traffic. The saving per ton-mile obtained by applying this unit saving to the estimated annual ton-miles of traffic, the estimated annual charges, and the ratio of annual benefits to annual charges for enlarging the existing channel to 18' x 125' are given in table 7:

TABLE 7

ECONOMIC ANALYSIS
(Enlargement from 12' x 125' to 18' x 125')

Section of waterway	Estimated annual charges	Estimated annual benefits	Ratio of benefits to cost
Port Arthur to Houston Ship Channel	$114,000	$318,000	2.8
Houston Ship Channel to Corpus Christi Bay	285,000	399,000	1.4
Corpus Christi Bay to Port Isabel	303,000	22,000	0.1

During the studies of channel enlargement, measurements were made of the actual speed of a tow moving through the Gulf Intracoastal Waterway, the Sabine-Neches Waterway, and the Lake Charles Deepwater Channel to determine the effect of channel size under operating conditions. The tow consisted of a towboat with 750 horsepower and two barges 240 feet long by 50 feet wide. The barges were loaded with 3,500 cubic yards of shell on each barge to a draft of 8.5 feet. The tow moved from the Houston Ship Channel to Lake Charles, Louisiana. A careful log of the trip was kept by observing the

engine speed and the time of passing known objects on the bank, such as mile posts, bridges, wharves, and similar objects. The travel distance was obtained by measuring the distance between the objects on maps, and from the reference line stationing where available. Observations were carefully analyzed and certain reaches that were more definitely identifiable were selected as test sections. These sections varied from 6 to 25 miles in length. Throughout the observations, the engine speed of the towboat was kept at 700 R.P.M. The sizes of the several sections of the waterway traversed in the tests were as follows:

Section of waterway	Depth and width of channel
Gulf Intracoastal Waterway	12' x 125'
Sabine-Neches Waterway	36' x 400'
Lake Charles Deepwater Channel	18'(1) x 100'

(1) Average over bottom width of 100'

The observed speed of the tow is shown in the following tabulations:

Section of waterway	Speed of tow loaded (miles per hour)
Gulf Intracoastal Waterway	5.6
Sabine-Neches Waterway	7.7
Lake Charles Deepwater Channel	6.8

It will be noted that the increase in speed amounted to 1.2 miles an hour in the Lake Charles Deepwater Channel and 1.9 miles an hour in the Sabine-Neches Waterway. These tests were not corrected for currents in the waterways or for wind. The results show a definite increase in tow speed in the larger channels that is the same magnitude as the increase derived from the analysis of the tow performance curves.

Since the analysis showed the saving would be the same for both of the typical tows, the estimates of saving for the additional sizes of channels also would be the same for either type of tow. The savings in cost of operation per ton-mile for the several enlargements, as shown by the curves on plate 2, were found to be as follows:

Channel enlargement	Estimated saving in ton-mile cost
12' x 125' to 18' x 125'	0.41 mills
12' x 125' to 18' x 200'	0.48 "
12' x 125' to 18' x 250'	0.56 "
12' x 125' to 18' x 300'	0.61 "

The total annual saving obtained by applying the saving per ton-mile to the estimated annual ton-miles of prospective commerce on the waterway, the estimated annual charges, and the ratio of benefits to costs for the four enlargements of the waterway are shown for the two reaches from Port Arthur to Corpus Christi in table 8. (Since the 18' x 125' enlargement of the channel from Corpus Christi to Port Isabel had so little justification, no studies of further widening of that section are shown.)

TABLE 8

ECONOMIC ANALYSIS
ENLARGEMENT OF GULF INTRACOASTAL WATERWAY

Section of waterway and enlarged channel size	Estimated annual benefits	Estimated annual charges	Ratio of benefits to cost
Port Arthur to Houston Ship Channel			
18 x 125	$318,000	$114,000	2.8
18 x 200	372,000	264,000	1.4
18 x 250	434,000	356,000	1.2
18 x 300	472,000	455,000	1.0
Houston Ship Channel to Corpus Christi Bay			
18 x 125	399,000	285,000	1.4
18 x 200	467,000	809,000	0.6
18 x 250	545,000	1,251,000	0.4
18 x 300	594,000	1,694,000	0.3

The problem of obtaining rights-of-way for any enlargement of the waterway has a material bearing on the feasibility of the project. Preliminary contacts with the local agencies along the route of the waterway indicate these agencies are not, generally favorable toward furnishing the extensive rights-of-way that would be required for the widening of the project. It is anticipated that there would be considerable difficulty in obtaining rights-of-way for the plans that call for extensive widening of the channel. Deepening of the channel to 18 feet with a bottom width of 125 feet would result in widening at the water surface of the present canal by 36 feet; at the existing 12-foot depth the deepening would widen the channel also by 36 feet.

At this time, the studies of the Corps of Engineers are in progress and the data presented in this report are subject to possible revision. The estimates of costs and benefits are preliminary in nature and are subject to modification before they are included in any report on this waterway that may be made by the Chief of Engineers to Congress. For these reasons, no conclusions have been presented in this paper.

ACKNOWLEDGMENTS

The writer as District Engineer of the Galveston District, Corps of Engineers, under whom the investigation was conducted, has general supervision over the studies covered in this paper.

N
TEXAS
LA.
Trinity River
Neches R.
Sabine R.
ORANGE
BEAUMONT
265
PORT ARTHUR
300
Brazos River
HOUSTON
SABINE NECHES WATERWAY
Galveston Bay
PORT BOLIVAR
Colorado River
San Bernard River
TEXAS CITY
GALVESTON
FREEPORT
400
FLOODGATES
PALACIOS
NAVIGATION LOCKS
VICTORIA
PORT LAVACA
Matagorda Bay
500
San Antonio Bay
ROCKPORT
Aransas Bay
ARANSAS PASS
PORT ARANSAS
CORPUS CHRISTI
Encinal Peninsula
Gulf of Mexico
600
Laguna Madre
PORT MANSFIELD
LEGEND
GULF INTRACOASTAL WATERWAY
TRIBUTARY CHANNELS
690 MILES FROM HARVEY, LOUISIANA
HARLINGEN
PORT ISABEL
Brazos Island Harbor
690
Rio Grande
BROWNSVILLE
MEXICO
SCALE OF MILES
25 0 25 50

PLATE I

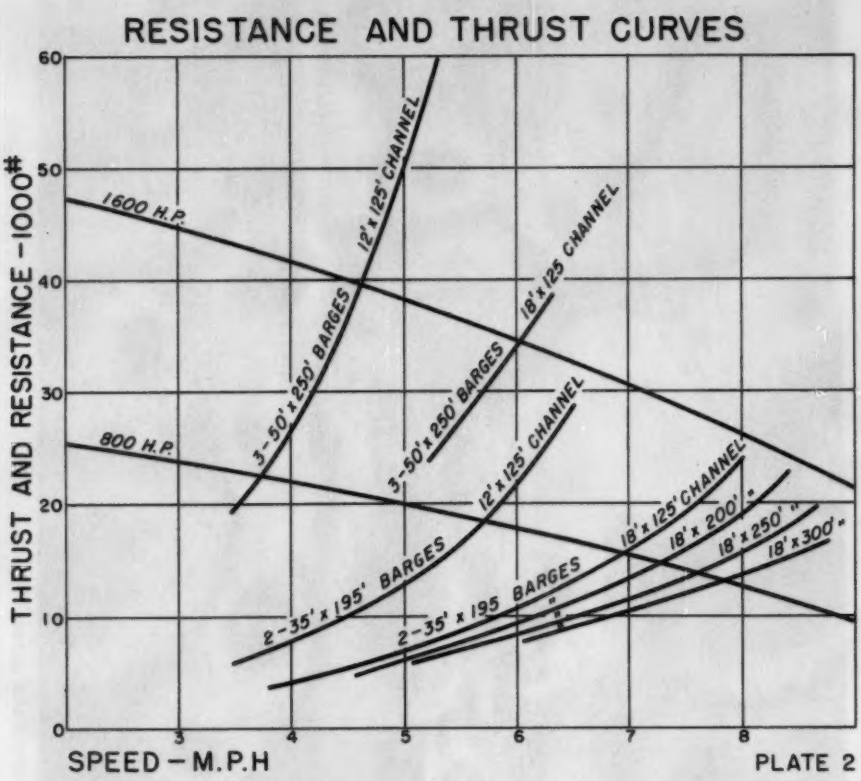

RESISTANCE AND THRUST CURVES
THRUST AND RESISTANCE - 1000#
SPEED - M.P.H
1600 H.P.
800 H.P.
12' x 125' CHANNEL
3 - 50' x 250' BARGES
18' x 125 CHANNEL
3 - 50' x 250' BARGES
12' x 125' CHANNEL
2 - 35' x 195' BARGES
18' x 125' CHANNEL
18' x 200' "
18' x 250' "
18' x 300' "
2 - 35' x 195 BARGES
0
10
20
30
40
50
60
3
4
5
6
7
8

PLATE 2

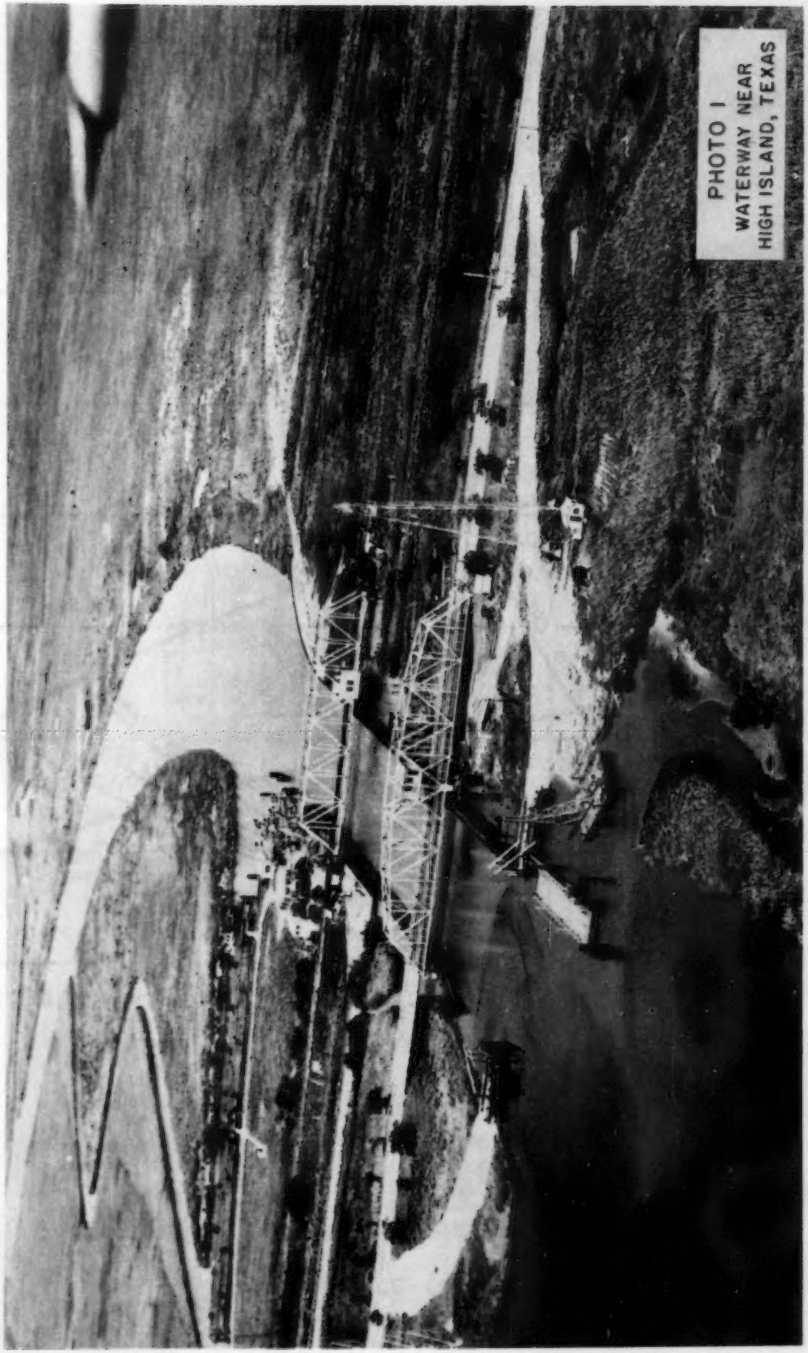


PHOTO I
WATERWAY NEAR
HIGH ISLAND, TEXAS

PHOTO 2
CROSSING OF BRAZOS RIVER
NOTE: FLOODGATES

PHOTO 3
LOCKS AT CROSSING OF COLORADO RIVER
(VIEW TOWARD EAST ALONG CANAL)

JOURNAL
WATERWAYS AND HARBORS DIVISION
Proceedings of the American Society of Civil Engineers

DESIGN PROBLEMS INVOLVED IN PROTECTION FROM TSUNAMIS

Kenneth Kaplan*
(Proc. Paper 968)

In the spectrum of ocean waves, tsunamis, or earthquake generated waves, fall midway between ordinary wind waves, and the tides. Periods of wind waves range from 5 to 20 seconds; the period of tides is about 12 hours; the Pacific Ocean tsunami of April 1946 had a period range of 12 to 15 minutes. This "betweenness" of tsunamis leads to relatively unique problems in designing for protection against them. The principles of protection against all types of ocean waves are the same; the details, however, can differ.

The stunami of 1946 originated off the Aleutian Islands, moved across the Pacific with speeds up to 400 miles per hour, and hit the Hawaiian Islands causing tremendous damage. For example, the town and port of Hilo, on the northeast side of the island of Hawaii, with a population of less than 20,000, received property damage estimated at 17 million dollars, and, in addition, 96 lives were lost.

Another tsunami in 1952 caused half a million dollars worth of damage at Hilo. Though it was smaller than the 1946 tsunami, the much smaller property damage figure was in large part due to the after effects of 1946, rather than the size of the waves in 1952. Most shorefront property had been destroyed in 1946, and rebuilding within a block of the shoreline had been forbidden.

Following these tsunamis, the Corps of Engineers initiated studies of means to protect Hilo from recurrent damage. As part of these studies, the Beach Erosion Board was asked to conduct wave tank investigations of tsunamis in general, and of means to protect against them.

This is a report on these tests.

It was first necessary to decide whether tsunami waves could satisfactorily be brought into the laboratory. A true small scale version was clearly out of the question. Tsunami waves are so long relative to their height that direct scaling in ordinary tank tests is virtually impossible. In 1946, for example, it is estimated that in deep water, the waves were only two-feet high, yet they were almost 100 miles long(1,2)**—more than 200,000 times longer than high.

Note: Discussion open until October 1, 1956. Paper 968 is part of the copyrighted Journal of the Waterways and Harbors Division of the American Society of Civil Engineers, Vol. 82, No. WW 3, May, 1956.

* Research Assoc., Broadview Research and Development, Burlingame, Calif.; formerly, Research Engr., Beach Erosion Board, USCE, Washington, D. C.

** Numbers in parentheses refer to the bibliography at the end of the report.

However, if tsunami type waves could be produced, and investigated over a wide range of conditions, it was possible that general relationships could be established which could be applied to the prototype case even without the use of accurately scaled tsunami models.

There were two characteristics of the 1946 tsunami which were considered important for these tests: First, that it was probably caused by a single displacement of the earth's crust;(2,3) second, that though many reports indicated that the first observed effect was a lowering of water level, tide gage records show that the first actual effect was a rise in water level. For example, in the Honolulu tide gage record for that period, reproduced in Figure 1, the first noticeable effect is a rise of about 1/2 foot.

These characteristics could be reproduced, in gross form, by a single positive displacement of water—positive because a negative displacement always generates a trough first, followed by a series of crests and troughs. A positive displacement always generates a crest first (which was desired)—sometimes, though not always, followed by a series of troughs and crests.

In Figure 2, tracings of waves caused by both negative and positive displacements are shown for purposes of comparison. Most records of waves used in these tests resembled those on the right in this figure, with the first crest being the highest. This characteristic was observed at Hilo during the 1952 tsunami.(4)

After the wave type was decided on, it was necessary to determine whether, with these waves, tank conditions could be established which could be related to prototype conditions. Both the desire for generalized results and scaling difficulties prohibited modeling precise hydrographic features. Therefore this question was investigated using continuous slopes as generalized representations of hydrography near a shoreline. These tests proved satisfactory.

Two types of protective barriers were then installed at the shoreline and their relative effectiveness found. The two types used were a "dike" type barrier (a 1 on 2 slope) similar to those ordinarily used against very long period water level changes such as the tide; and a reflecting type barrier (a vertical wall) to simulate ordinary seawalls, the usual protection against wind waves.

Finally the barriers were removed, and the characteristics of the waves themselves at the shoreline were investigated.

In Figure 3 is shown the complete experimental setup.

The upper portion of the figure illustrates conditions just prior to generation; the lower portion conditions just following generation. The bulkhead was connected to a varidrive driven disk with a spring loaded pin, then disconnected from the disk at the completion of a forward stroke. Connecting the pin to various points on the disk allowed variation in the length of stroke, and changing the varidrive setting allowed variation in the speed of the stroke. Two closely spaced wave gages were installed at the toe of the slope, both to measure wave height in deep water (which in these tests was necessarily the deeper part of the tank) and to enable wave length determinations to be made. At the shoreline, linear scales were mounted on both the slopes and the barriers, and wave runup was measured by eye. In all cases, only the first wave of any group was used, its deep water length for these tests being defined as twice the horizontal distance between the first measurable water rise, and the first crest.

The reasons for using only the first wave, even though in 1946 there were many waves higher than the first are of particular interest. Preliminary tests

showed that succeeding waves of any wave group not preceded by a trough would run up no further than would the first wave of the group, even though they were more than twice the height of the first wave. Therefore, by utilizing only the first wave, that wave which would give the greatest value of runup for any given height was being investigated.

Two long slopes were used. The first, a 1:30 slope, was found to be too steep for even these generalized studies to apply to Hilo's problem. No loss of generality was involved in installing a flatter 1:60 slope, which more nearly fit the conditions desired, and it was on this slope that the barriers were installed. For these tests, only runup, and deep water wave height and length were measured. From these, the dimensionless parameters,

$\frac{\text{Runup}}{\text{Wave Height}}$, (relative runup), and $\frac{\text{Wave Height}}{\text{Wave Length}}$, (wave steepness) were computed.

In all over 1000 runs were made.

For the four tests, those on two slopes and the two types of barriers, R/H was plotted against H/L on log—log paper. The results are shown in Figure 4.

Plotted in the four cases is a least squares computed line, and to show the consistency of the data, (which was very good), a probable error envelope.

Though the 1:30 slope was eventually found to be too steep for results on it to be applied to Hilo, these results, plotted in the upper left on Figure 4, were of prime importance to all subsequent tests. The points for the curve were measured with three different tank water depths. Yet the three depths gave virtually the same curve. It was reasoned from this that if the depth of measurement of H and L made no difference in the determination of relative runup, then tank results could be compared with prototype data.

Unfortunately, to compare tank and prototype data it was necessary to extrapolate the curves over three more cycles of H/L. This extrapolation is probably the weakest point of the whole series of tests, but because of the impractibility of modeling tsunami waves little else could be done. The upper points on each plot represent the limit values of H/L which could be generated. Certain internal checks were made which indicated that the calculations were not grossly in error but, any extrapolation is weak.

The results of the barrier tests can be better examined if they are plotted together as in Figure 5.

The least squares line for both walls are very close, but the data for the reflecting wall shows a much greater variability than that for the dike. This suggests (though more tests would be necessary to confirm or deny the conclusion) that a dike or levee with a fairly steep slope would on the average be as effective against tsunami runup as a reflecting type wall such as many ordinary seawalls. Reflecting type walls with crests low enough to permit passage of some water over them can, however, be more effective in allowing no passage of water past the shoreline than dike type walls of equal height.(5)

The results of the barrier tests when applied to Hilo indicated that a runup of twice the shoreline wave height was possible. Previously a 15-foot high wall was designed on the basis of a runup of one wave height. Rough overtopping, as opposed to runup, computations similar to those used in the design of the 15-foot wall were made using twice the shoreline wave height. These indicated that though overtopping quantities might be larger than those used in the wall design, the 15-foot high wall might still be considered reasonably effective against overtopping.

The preceding essentially describes the portion of the testing program from which immediately usable data were derived. Well defined relationships between the dimensionless parameters $\frac{\text{Runup}}{\text{Wave Height}}$ and $\frac{\text{Wave Height}}{\text{Wave Length}}$ were developed, which seemed to be independent of the depth of measurement of wave height and length. With these—admittedly utilizing a large extrapolation and an approximate method of overtopping computation—results were found which, with reservations, were applicable to the Hilo problem.

The testing program as a whole, however, pointed up two additional items of interest. As mentioned previously the program was extended to investigate wave characteristics at the shoreline, that is, breaking waves. Quantitative measurements of wave parameters, specifically wave height, showed a bias believed to be due to instrumental limitations. These quantitative results, therefore, could not be used in the program. During these tests, however, one surprising wave characteristic was noted. The waves recorded were breakers. It was anticipated that the turbulent characteristics of these waves were such that they could only rarely be reproduced. Yet the reproducibility of wave shape was consistently excellent. Illustrative of this are the wave records reproduced on Figure 6.

The records of four separate, but paired, runs are shown. Generator setting and water depth for the two on the left were the same; a different setting and depth were used for the two on the right. Note that the wave profile for each pair are virtually identical.

A second item of interest developed from the testing program, but was investigated outside it. The characteristics of the generated wave train changed considerably with change of water depth and generator setting. Most often a train of five, six, or seven waves developed. At one particular setting a single wave, very much like a solitary wave developed. And at one other setting, surprisingly, two separate and distinct waves formed. After removal of the slopes and barriers, the water depth and generator settings for this "double wave" train were reset. The waves were permitted unimpeded reflection between the far tank wall and the generator bulkhead. Records were taken at various distances from the point of generation, a sampling of which are shown on Figure 7.

The waves retained their dual nature until too small to measure. This took as many as five complete transits of the tank, or distances equivalent to over 1000 feet from the generator bulkhead.

BIBLIOGRAPHY

1. Shepard, F. P., G. A. MacDonald and D. C. Cox, "The Tsunami of April 1, 1946," Bull. of the Scripps Inst. of Oceanography, Vol. 5, No. 6, pp. 391-528.

2. Munk, W. H., "Report on Protection of Hilo, Hawaii, from Tsunamis," Report to District Engineer, Honolulu District, Corps of Engineers, 1950 (unpublished).

3. Byerly, Perry, Appendix to report by M. P. O'Brien, "Preliminary Report on Seismic Sea Waves from Aleutian Earthquake of April 1, 1946," Univ. of Calif., HE-116-207, April 25, 1946, (unpublished).

4. Letter from Honolulu Area Engineer to San Francisco District Engineer dated 25 November 1952, subject: Tsunami of 4 November 1952.

5. Saville, T., Jr., and J. M. Caldwell, "Experimental Study of Wave Overtopping on Shore Structures," Proceedings, Minnesota International Hydraulics Convention, September 1953.

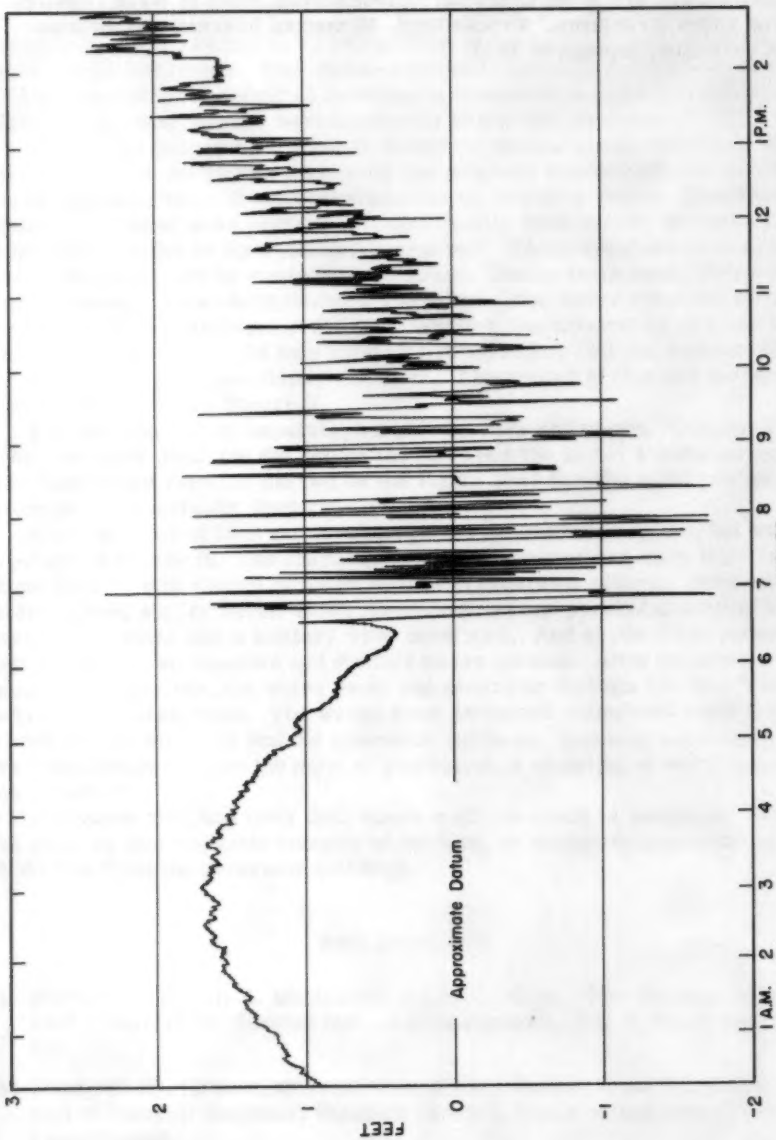


Figure 1. Record of the tsunami of April 1, 1946 on the tide gage in Honolulu Harbor. (Courtesy USC & GS)

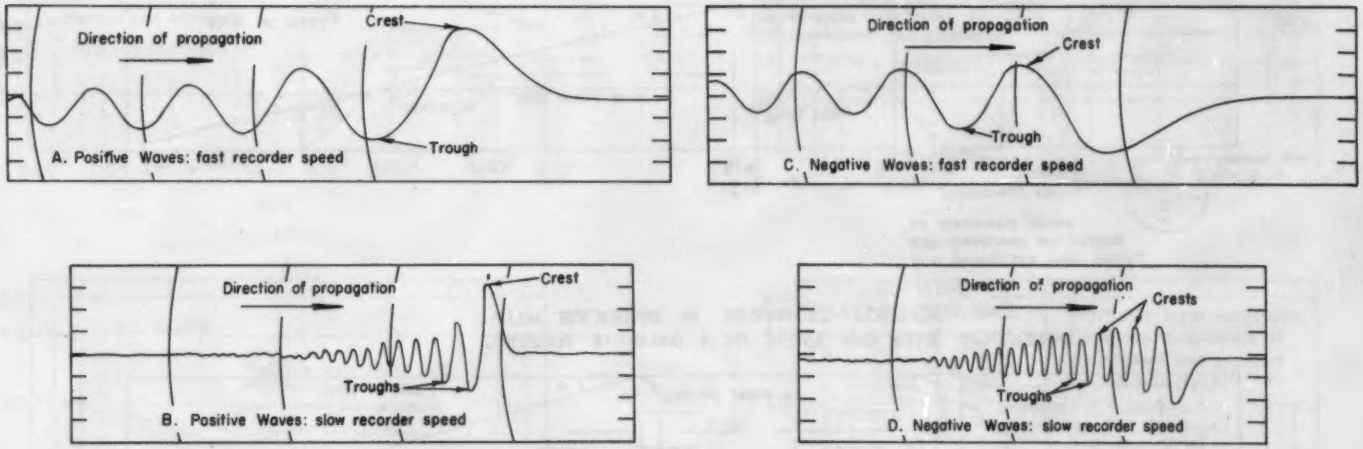


Figure 2. Records of positive and negative waves.

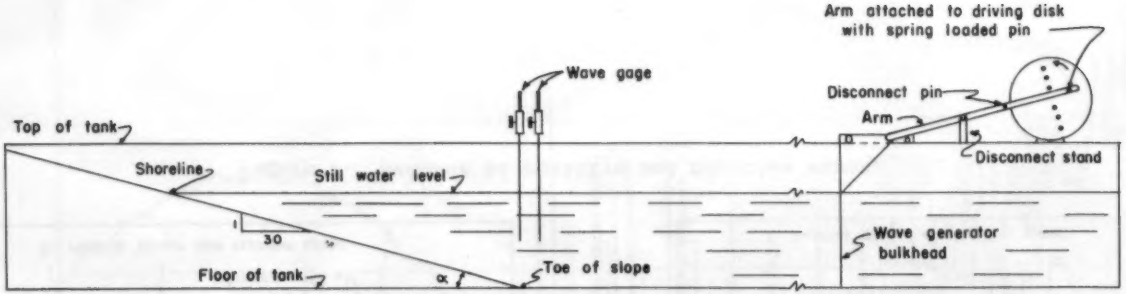


DIAGRAM SHOWING 1:30 SLOPE AND WAVE GENERATOR WITH BULKHEAD IN REARMOST POSITION

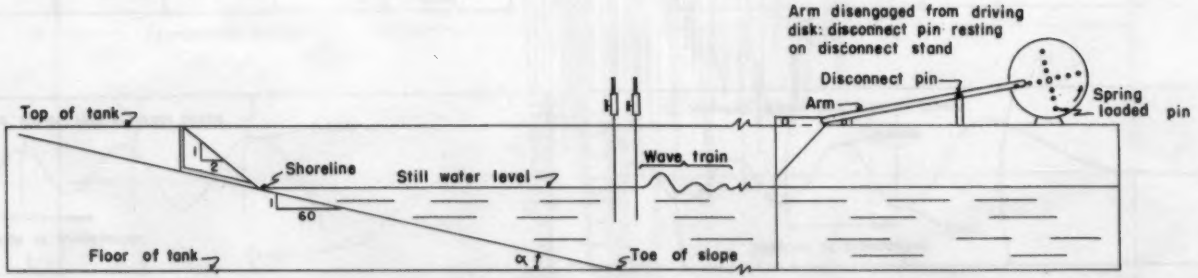


DIAGRAM SHOWING 1:60 SLOPE WITH "DIKE" TYPE WALL AT THE SHORE LINE AND WAVE GENERATOR WITH BULKHEAD IN FOREMOST POSITION

Figure 3. Sketches of laboratory test installations.

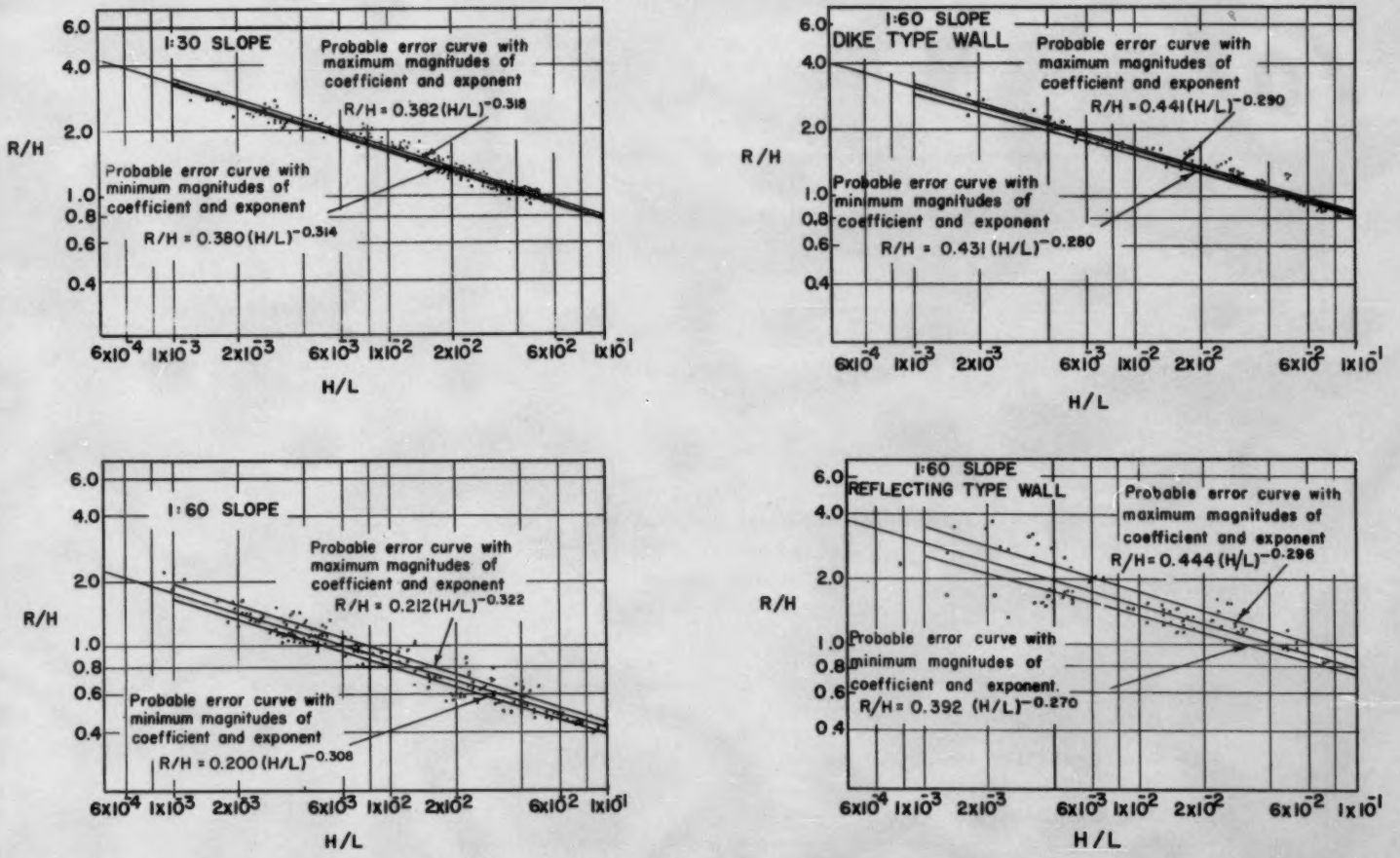

1:30 SLOPE
Probable error curve with maximum magnitudes of coefficient and exponent
R/H = 0.382(H/L)^-0.318
Probable error curve with minimum magnitudes of coefficient and exponent
R/H = 0.380(H/L)^-0.314
1:60 SLOPE
DIKE TYPE WALL
Probable error curve with maximum magnitudes of coefficient and exponent
R/H = 0.441(H/L)^-0.290
Probable error curve with minimum magnitudes of coefficient and exponent
R/H = 0.431(H/L)^-0.280
1:60 SLOPE
Probable error curve with maximum magnitudes of coefficient and exponent
R/H = 0.212(H/L)^-0.322
Probable error curve with minimum magnitudes of coefficient and exponent
R/H = 0.200(H/L)^-0.308
1:60 SLOPE
REFLECTING TYPE WALL
Probable error curve with maximum magnitudes of coefficient and exponent
R/H = 0.444(H/L)^-0.296
Probable error curve with minimum magnitudes of coefficient and exponent.
R/H = 0.392(H/L)^-0.270
R/H
H/L
6.0
4.0
2.0
1.0
0.8
0.6
0.4
6x10^-4 1x10^-3 2x10^-3 6x10^-3 1x10^-2 2x10^-2 6x10^-2 1x10^-1

Figure 4. Relative run-up (R/H) vs. wave steepness (H/L).

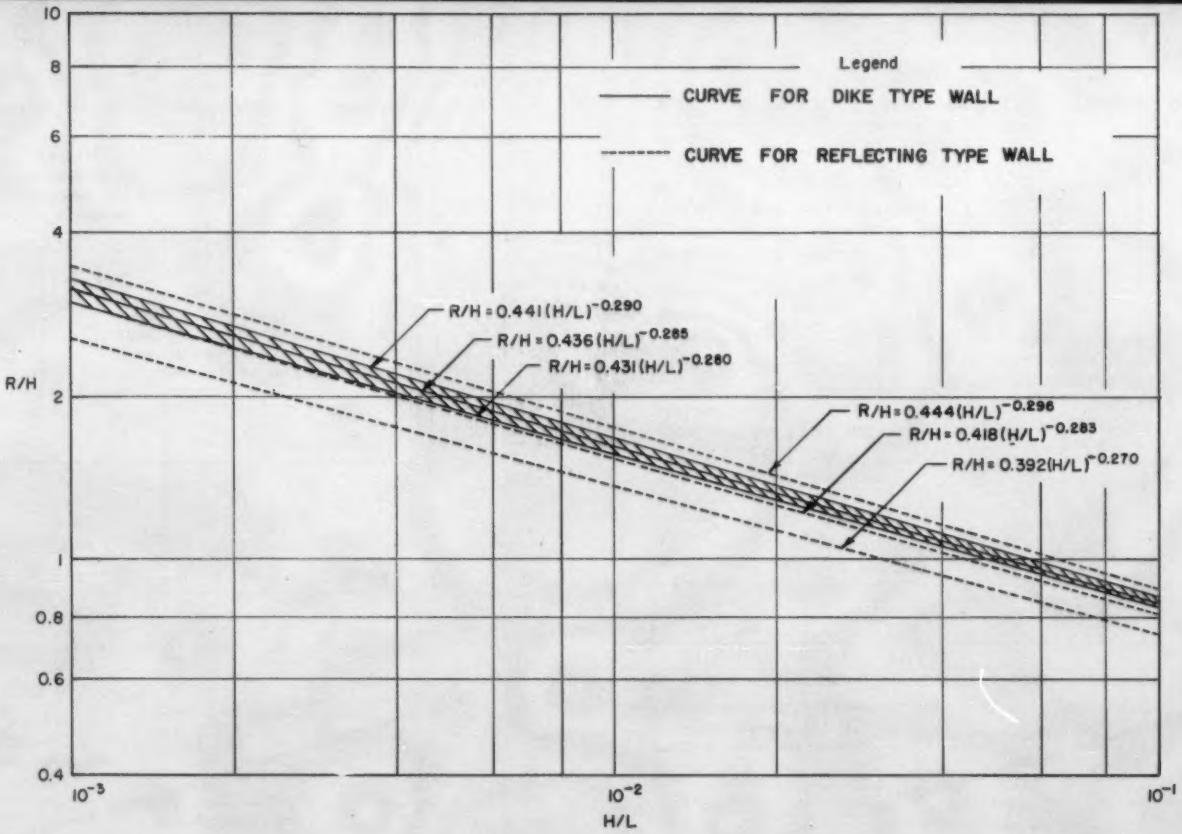
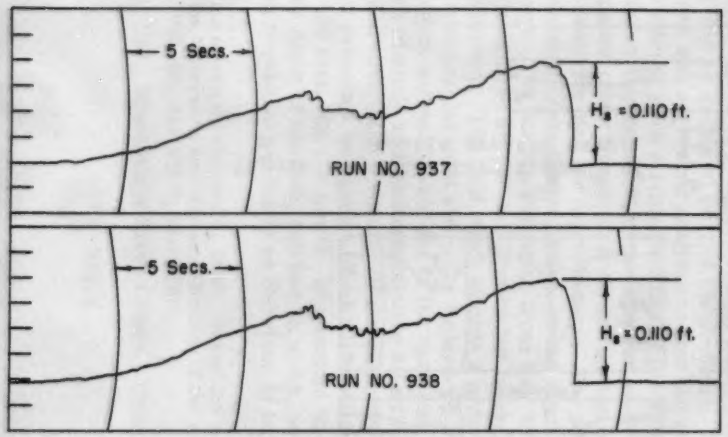
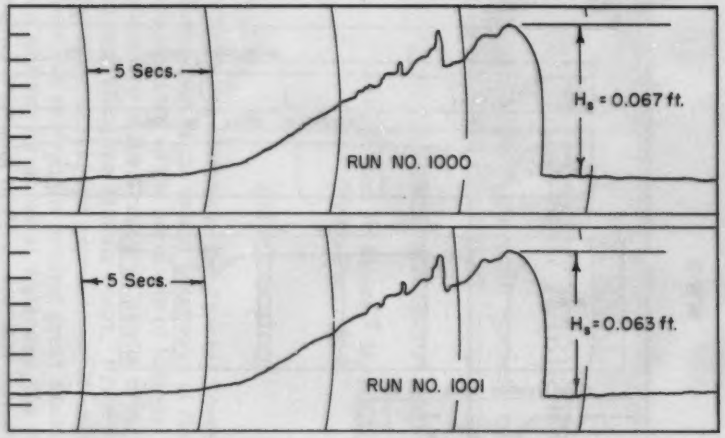


Figure 5. Comparison of (R/H) vs (H/L) curves for "dike" and "reflecting" type walls at the shoreline of a 1:60 slope.



Run 937 & Run 938: Generator Stroke = 23 inches; Generator Speed Setting = 0.37 rev./sec.; Depth at the toe of the slope = 0.9 feet



Run 1000 & Run 1001: Generator Stroke = 15 inches; Generator Speed Setting = 0.37 rev./sec.; Depth at the toe of the slope = 0.5 ft.

Figure 6. Typical laboratory shoreline wave records.

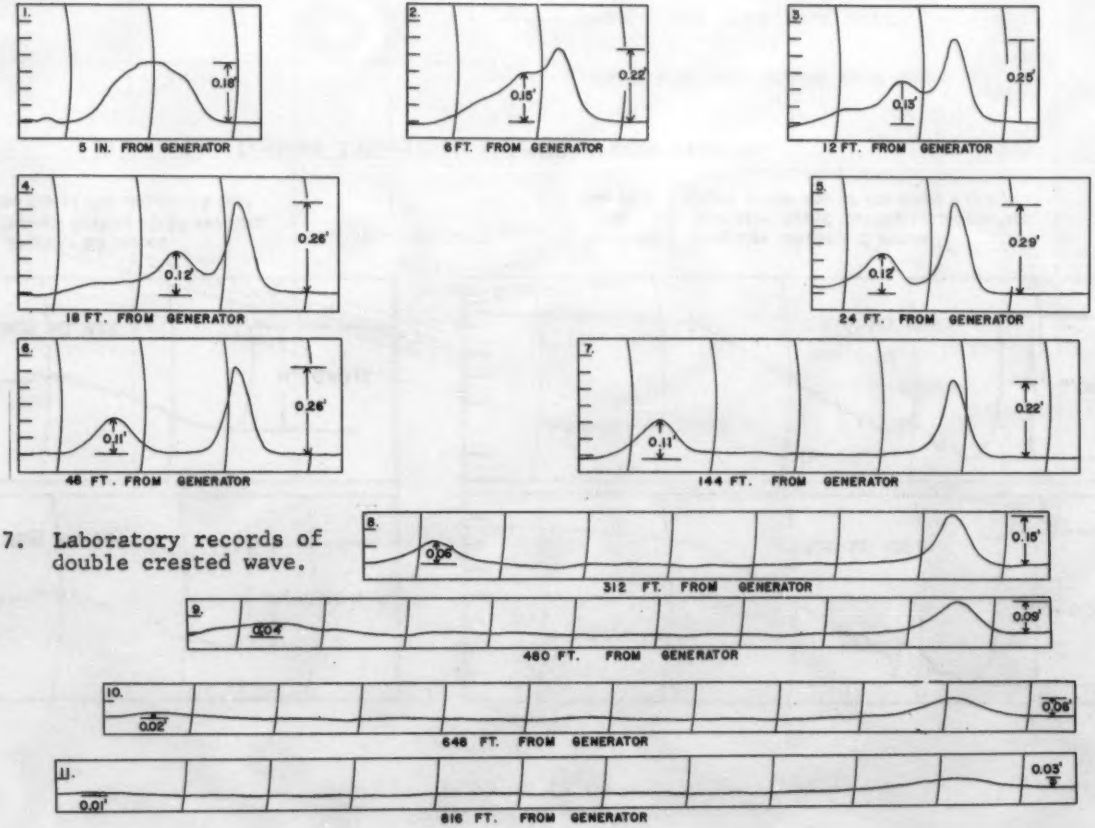


Figure 7. Laboratory records of double crested wave.

JOURNAL
WATERWAYS AND HARBORS DIVISION
Proceedings of the American Society of Civil Engineers

NEW LOCKS ON LOWER MISSISSIPPI AND GULF COAST

Norman R. Moore,[1] M. ASCE
(Proc. Paper 969)

INTRODUCTION

One of the earliest activities of the Federal Government was the development and improvement of the inland waterways of the United States. Reporting to the Senate in 1808 on the necessity of National improvement by roads and canals, Secretary of the Treasury Gallatin proposed, among other works, an inland waterway along the Atlantic coast. Prior to that date there had been scattered local construction and improvement of waterways. From such early beginnings there has evolved the present great Mississippi River system which connects with the Intracoastal Waterway, the Great Lakes, and other inland waterways. Pursuant to Congressional authority, Federal investigation of the Mississippi River was begun in 1820. Improvement of navigation was the original objective of these studies, but it was soon recognized that flood control was an essential part of the problem. Moreover, following the Civil War there was a growing realization that the major flood control and navigation problems were a responsibility of the Federal Government. The Mississippi River Commission, created by Act of Congress in 1879, was authorized, among other things, to study navigation on the Lower Mississippi River and to submit a plan for improvement. The portion of the Intracoastal Waterway extending from Boston, Mass. to the Florida Keys is known as the Atlantic Section. The portion extending from Key West to the Mexican border is officially designated as the Gulf Section but is usually termed the Gulf Intracoastal Waterway. The Gulf route affords a protected, safe, all-weather waterway to commercial tows and other light-draft vessels that are not designed to navigate extensive reaches of the open sea.

Many years of study and much legislative effort have resulted in the presently authorized navigation projects. When completed, the Gulf Intracoastal Waterway will provide a navigation channel 12 feet deep with a minimum bottom width of 125 feet. On the Lower Mississippi River, the authorized 12-foot low-water depth of channel, 300 feet in width, between Baton Rouge and Cairo is being developed gradually as the river is improved, confined, and

Note: Discussion open until October 1, 1956. Paper 969 is part of the copyrighted Journal of the Waterways and Harbors Division of the American Society of Civil Engineers, Vol. 82, No. WW 3, May, 1956.

1. Chief, Eng. Div., Mississippi River Comm., Vicksburg, Miss.

revetted. Between Baton Rouge and the Gulf of Mexico, the authorized channel depths vary from 35 to 40 feet, and the width of channel ranges from 500 to 1,500 feet, the maximum width being within the limits of the Port of New Orleans. On the Atchafalaya River, the channel will be 12 feet in depth below mean low water on a bottom width of 125 feet from Morgan City to the Mississippi River via the Atchafalaya and Old Rivers, and from the -20-foot contour in Atchafalaya Bay to the -20-foot contour in the Gulf of Mexico, the project channel is 20 feet deep and 200 feet wide. The existing channel of the Atchafalaya River provides sufficient depth between Morgan City and Atchafalaya Bay.

The portions of these waterways with which this paper is concerned include the Mississippi River from Old River (mile 301) to the Passes, the Red, Old, and Atchafalaya Rivers, and the Gulf Intracoastal Waterway within the State of Louisiana, together with interconnected waterways. (Figure 1) These channels offer a natural and economical means of transportation for the extensive variety of commodities produced or consumed in the Gulf region.

Character of Traffic

Between Baton Rouge and the Gulf, a large part of the total tonnage of traffic is carried in self-propelled vessels. Over the other waterways considered herein, by far the greater amount is carried in steel barges, which are moved by high-powered tugs or towboats. These barges are of various dimensions, ranging from 26 to 50 feet in width and from 150 to 300 feet in length. A tow is ordinarily made up of several barges tied together by means of steel cables and ratchets to form a single rigid unit which is pushed ahead of the towboat. Infrequently, the barges are pulled behind the tug on a hawser. The number of barges in a tow and their arrangement depend in part on the alignment and width of the waterway. The design of a lock on these waterways must therefore take into consideration not only the prevailing tow make-up but the changes that may reasonably be expected to develop during the economic life of the project.

Requirements for Locks

On the Lower Mississippi River, the slope of the stream is relatively flat at all stages and hence navigation locks are not necessary. But such structures are necessary to overcome the difference between the water surfaces where navigation is required to cross levee lines along rivers or floodways into protected areas, or into sea-level canals. Locks may also be needed to protect agricultural lands from salt water intrusion by controlling tidal flows.

Early Lock Construction

As a result of Congressional authorization of a waterway from Plaquemine to Grand Lake, La., construction of a lock connecting Bayou Plaquemine and the Mississippi River was begun in 1895 and completed in 1909. This structure, which is still in use, has a usable length of 260 feet, a chamber width of 55 feet, a depth over the miter sill of 10 feet, and a maximum lift of about 34 feet. The gates were originally operated by steam power, but are now electrically controlled. This lock must be closed to traffic during high stages of the Mississippi River due to the difficulty of operating the gates.

Still earlier, about 1880, a lock was begun at Harvey, across the river from New Orleans. The initial project was evidently not completed and for a number of years, the only facility at this point was a lock gate which could be used only when the canal and river were at approximately the same level. In 1910, the report of a Special Board of Engineers noted that the lock at Harvey had a width of 29 feet 11 inches, an available length of 150 feet, and a depth of 7 feet over the miter sills. This lock connected to a private canal. Possession of these works was acquired by the United States in 1924 and, soon thereafter, construction of the present Harvey Lock was authorized.

Similar structures built on privately owned waterways early in the twentieth century have either been superseded by modern structures, have fallen into disuse, or in some cases have been abandoned.

Design Characteristics of Modern Locks in This Area

The chief problem presented to the designer of a lock within the Lower Mississippi River and Gulf Coast region is that of foundation for a structure of this type on the alluvial and marine deposits prevalent in the area. Of prime importance in many instances is the necessity of assuring security against underseepage.

Because they will withstand a head in either direction and can be opened or closed while under head, sector gates have been selected in many cases despite a somewhat greater initial cost than for miter gates. For relatively low head differentials, the use of sector gates makes it unnecessary to provide valves and conduits for filling and emptying the lock. Among other structural and hydraulic advantages are the substantial reduction in live load reaction from the gate to the masonry in comparison with that of miter gates, and the advantage in being able to use the lock structure as a sluiceway. With such gates, filling and emptying of the lock chamber is effected by partial openings of the gates permitting flow between the two gate sectors and also in some cases between the skin plate and the gate recess in the masonry. In order to study the filling operation and to ascertain the proper speeds for gate operation to control turbulence in the lock chamber and the magnitude of hawser stresses in the mooring lines of tows, model studies of sector gates have been conducted at the Waterways Experiment Station.

For some of the structures the required lifts are quite high, resulting in high walls in gate bays and lock chambers, although in no instances are the resulting structures unprecedented.

Recent Lock Construction - Algiers Lock

As previously mentioned, construction of a new lock at Harvey, La., across the Mississippi River from New Orleans, was authorized following acquisition by the United States in 1924 of the privately owned lock and canal. The structure, completed and placed in operation in 1933, has a usable length of 425 feet, a chamber width of 75 feet, and a depth on the miter sill at low water of 12.0 feet. It is designed for a maximum lift of 19.6 feet which is adequate for the range of river stages to be expected at this point on the lower River. During and after the second World War, barge traffic through this lock increased to such an extent that operation of the lock at its maximum capacity twenty-four hours a day was imperative. Even under such an operating schedule, tows are frequently required to wait for extended periods to be passed through

the main line levee. In order to make repairs to the lock, it is necessary to shut down all traffic, and necessary maintenance operations cannot be deferred until the closed navigation season because there is none in this latitude. Consideration was therefore given to construction of a second lock, so located as to divide the load with Harvey Lock and to relieve some of the congestion in New Orleans Harbor. The site selected for this structure is downstream from New Orleans at Algiers, about 2,000 feet from the Mississippi River. The new canal from the Algiers Lock connects with the Intracoastal Waterway at Mile 6 from Harvey Lock, has a length of about 9 miles, and provides a navigable channel 12 feet deep and 125 feet wide. This new facility has a chamber width of 75 feet and a usable length of 760 feet. The depth over the miter sills is 13 feet. The structure is designed for a maximum lift of 18 feet and will be usable at all anticipated stages of the Mississippi River.

The foundation design of this structure presented an unusual problem because gas pressure in an underlying sand stratum with surface at elevation -54 feet, when combined with the estimated hydrostatic uplift, exceeded the weight of earth between the top of this sand formation and the bottom of the required excavation. The solution consisted of lowering the pressure by means of gas and hydrostatic relief wells. The lock structure is founded on a relatively thin stratum of clay and silt in thin layers, below which is a stratum of soft clay having a thickness of about 29 feet. This compressible clay necessitated provision of a pile foundation to carry the superimposed load to a second sand layer below the clay, which is underlain at about elevation -70 feet by the stiff Pleistocene formation. The reinforced concrete structure is of the rigid frame or "dry-dock" type.

In the gate bays, the floors are 10 feet in thickness and the wall thicknesses vary from 2 feet at the top to 6 feet at the base. In the chamber, the floor is 6.5 feet thick and the wall thickness varies from 2 feet at the top to 5.5 feet at the base.

The necessity of making provision for reversed heads at this location suggested the use of sector gates. In the event of damage to the gates necessitating a shutdown of the lock, an emergency dam of the stop-log type is available for closure. The gates and all appurtenant equipment are electrically operated.

Construction of the Algiers Lock has been completed (Figure 2) but the facility has not been placed in service because of delays in completion of relocations. It is anticipated that the structure will be in operation early in 1956. The addition of this lock and its connecting canal to the Intracoastal Waterway will alleviate the congestion and consequent delays at Harvey Lock, and will also relieve traffic in the most heavily traveled portion of New Orleans Harbor.

Port Allen and Bayou Sorrel Locks

An alternative route between the Mississippi River and the Gulf Intracoastal Waterway from the west is provided through the Plaquemine to Morgan City waterway and the Plaquemine Lock, mentioned above, which is located a little more than 100 river miles above Harvey Lock. Recognizing the inadequacy of the present lock and the connecting 9 by 100-foot channel to the main Waterway at Morgan City, in 1946 Congress authorized enlargement of this existing channel from Morgan City to Indian Village to 12 by 125 feet and extension of this route to the same dimensions by way of Bayou Grosse

Tete and a new land cut about 18 miles in length to the Mississippi River at Port Allen, which is opposite Baton Rouge. The extension will serve as the main outlet for the drainage network of West Baton Rouge Parish and will serve the navigation needs of the petroleum and chemical industries of the Baton Rouge area as well as the needs of traffic from and to points beyond the termini of this Waterway.

A new lock is to be constructed in the levee at Port Allen. The lock now (1956) being designed is larger than any existing lock in the area. It will have a chamber 84 feet in width and 1200 feet long, with a 14-foot depth over the sills and a maximum lift of 44 feet. The length conforms to that of new locks being constructed or already completed on the Ohio and Upper Mississippi Rivers. The structure will be of the rigid frame or "dry-dock" type, without bearing piling. The gates will be horizontally framed steel miter gates, operated by an electrically controlled hydraulic system of the modified Ohio River linkage type. Longitudinal wall culverts each 14 feet square in cross section, provided with side ports, will be provided for filling and emptying of the chamber. Flow in the culverts will be controlled by individually operated segmental valves of the reversed tainter valve type. The outlet manifolds will be of the split flow type, discharging both into the lower approach chamber from each side and directly into the canal from outlet boxes located in the end walls. Model tests are in progress at the Waterways Experiment Station to study the design of the intake and outlet manifolds and the wall ports, and to study the relation of filling rates to hawser stresses.

Emergency closure of the lock at the river end and unwatering of the gate bays or chamber will be provided for by means of fabricated sectional steel bulkheads which will be manipulated by a stiffleg derrick and derrick boat.

Excavation of the channel extension from Indian Village to Morley is in progress as of 1956, and construction of the lock will be commenced upon appropriation of funds, which are anticipated in Fiscal Year 1957.

The Alternate Route crosses the East Atchafalaya Basin protection levee about 15 miles below Plaquemine. Although provision of a lock at the levee crossing was planned prior to 1940, construction was not undertaken until after the war, and the Bayou Sorrel Lock (Figure 3) was opened to traffic in 1951. The structure is of the earth-chambered type, which it was determined would be economical at this site. This type may not be desirable if the chamber is large, or if to meet stability requirements the chamber side slopes must be very flat, with the result that the chamber volume is so great that the filling and emptying times are excessive. Moreover, when this type of lock is used, the chamber side slopes must be revetted to protect the earth slopes from wave action and propeller wash, and a guide wall on one or both sides may be required to prevent grounding of tows from transverse winds. The maintenance costs of these features must be considered in determining the economy of this type of construction.

The lock consists of two reinforced concrete gate bays on timber bearing piles, with steel sector gates separated by an earthen channel with paved side slopes. It is provided with a timber guide wall on one side in the chamber and with similar walls for both approaches. The usable length is 760 feet and the clear width is 56 feet. The depth over the sills is 14 feet. The lock is designed for a maximum lift of 21 feet.

Difficulties experienced during the construction of Bayou Sorrel Lock emphasize the need for extreme care in making soil investigations and in carrying out excavation and fill operations in this area. During the excavation for

the lock chamber a slide failure occurred which was of such depth and magnitude as to require relocation of the lock. The failure was considered to be due principally to the construction slopes (1 on 3) used in this highly plastic, weak clay, but contributing factors probably were a levee constructed about 50 feet from the top of the excavation and the removal of a layer of humus in the bottom of the excavation. In the final construction of the chamber, in order to construct it with side slopes (1 on 2) which permitted reasonable filling and emptying periods, it was necessary to follow a rather involved procedure of overexcavation laterally at part depth, flooding of the lower part of the excavation during refill, and finishing to final section by refill in stages.

Bayou Boeuf Lock

The East and West Atchafalaya Basin Protection Levees constitute the limits of the Atchafalaya Basin Floodway, the function of which is to prevent excessive stages in the Mississippi River below the latitude of Old River by diverting through the Atchafalaya Basin to the Gulf those flood flows that would overtax the safe capacity of the Mississippi River. The East Atchafalaya Basin Protection Levee, in which the Bayou Sorrel Lock is constructed, begins at the lower end of the lower guide levee of the Morganza Floodway and extends southward through Morgan City to Cutoff Bayou, crossing Bayou Boeuf and the Gulf Intracoastal Waterway. Completion of the levee required to protect areas and communities east of Morgan City from floodwaters in the Atchafalaya Basin made necessary the construction of a lock in the levee to provide a navigation connection. As in the case of Bayou Sorrel Lock, the Bayou Boeuf Lock has an earth chamber. It is 1160 feet long and 75 feet in usable width, with a depth over sills of 13 feet at mean low Gulf level. The design provides for a maximum lift of 9 feet which is expected to be required rarely. The structure is provided with steel sector gates hydraulically operated with electrical controls. The gate bays bear directly on earth foundations. The earth chamber section is protected with stone riprap and articulated concrete mattresses. Timber guide walls are provided in the chamber and approach sections. Construction was initiated in 1952 and the lock was opened to traffic in 1954. (Figure 4) A bypass route around the lower end of the levee, through a dredged borrow pit and Bayou Chene, is provided as an alternative route during periods of high water in the Atchafalaya River when adverse flow conditions will make navigation difficult in the vicinity of the lock entrance.

Berwick Lock

The West Atchafalaya Basin Protection Levee, previously described as an important element of the adopted project for flood control of the Mississippi River and tributaries, begins near the town of Hamburg, Louisiana and extends in a southeasterly direction to the Wax Lake Outlet, and thence eastward to and through Berwick to the Intracoastal Waterway. The levee crosses the Lower Atchafalaya River just above Berwick Bay. Berwick Lock (Figure 5), completed in 1950, is provided at that point in order to permit navigation to pass up the Lower Atchafalaya River into Bayou Teche to reach Patterson and other communities. This concrete structure is 45 feet in width, has a sill elevation 9 feet below mean low Gulf, and has a usable length of 300 feet between steel sector gates, which is adequate in size for the small boats and single barges that comprise the local traffic using this waterway.

Calcasieu Lock

In western Louisiana, the Intracoastal Waterway traverses the Mermentau River Basin along the northerly edge of the marshlands bordering the Gulf of Mexico. In this basin area, extensive cultivation of rice requires the pumping of large amounts of fresh water from the streams and from the connecting Grand and White Lakes during the growing season of June through August. As the water levels fall during periods of low stream flow, salt water flows up the stream channels, the bottoms of which are below the elevation of Gulf level. In order to obstruct this ingress of salt water by way of the Intracoastal Waterway, a number of structures have been constructed in the area. On the east is Vermilion Lock, which blocks intrusion from Vermilion Bay. On the south are the Catfish Point and Schooner Bayou Control Structures, which are multiple floodgates of the sector type across the Mermentau River and Schooner Bayou, respectively. The most recently completed structure (1950) for this purpose is the Calcasieu Lock (Figure 6) which prevents the entrance of salt water into the basin from the Calcasieu River via the Intracoastal Waterway. It is located in a dredged land cut forming a relocation to the north of the former route of the Waterway.

The lock has an earth chamber with 1 on 2 side slopes, protected by articulated concrete revetment mats, which has a usable length of 1,180 feet, a width of 75 feet, and a depth over the sill of 13 feet. The gate bays are designed as rigid frames without foundation piling, and the sector gates, of welded structural steel, are electrically controlled. The lock is designed for a maximum differential head of 6.0 feet and a maximum reverse head of 2.0 feet. The lock chamber and approach guide walls are of timber. Needle dams are provided to permit unwatering for inspection and maintenance purposes.

Locks below New Orleans

An improved navigation channel 9 feet deep and 80 feet wide, designated as the "Waterway from Empire, La. to the Gulf of Mexico" extends from about Mile 29 Above Head of Passes via Doullut Canal and Bayous Long and Fontanelle to the -9-foot contour in the Gulf, a total distance of about 10 miles. This route has long been used by shallow draft boats engaged in the fishing and shrimping industry. Completion of the improvement by the Federal Government in 1950 opened the route to oil traffic between the oil fields of Plaquemine Parish and the refineries along the Mississippi River and the Intracoastal Waterway and provided a direct route for deeper draft fishing craft operating between the Mississippi River and fishing grounds west of the mouth of the river. The Federal improvement also made obsolete the former small shallow draft lock in Doullut Canal, and the difficulty of operation led to the construction by the State of Louisiana of a new lock. Empire Lock, completed in 1950, has a chamber width of 40 feet and a usable length of 197 feet 6 inches, with a depth over the sill below mean Gulf level of 10 feet. It is of reinforced concrete construction and has two sets of electrically operated steel sector gates in concrete gate bays. Needle beams are provided for emergency closure of the lock at each end.

Across the Mississippi River from the Empire Lock and about 4 miles by river downstream, a waterway extends from the river eastwardly to Quarantine Bay, a distance of a little more than a mile. For vessels of 5-foot draft and less, this channel makes possible a saving of approximately 49 miles of

navigation. Vessels of greater draft are afforded a saving of as much as 82 miles of navigation in the unprotected outside waters which are hazardous to navigation during heavy winds or fogs. Ostrica Lock, located at the Mississippi River end of this waterway, replaces an old small wooden lock which had been closed to navigation since the hurricane of 1947. It was completed in 1953 by the State of Louisiana. The design and dimensions are similar to Empire Lock except that Ostrica Lock has a usable length of 236 feet 6 inches.

Proposed Lock Construction

The project for control of Old River, authorized by Public Law No. 780, 83d Congress, approved September 3, 1954, provides for control of flows from the Mississippi River into the Atchafalaya River and Basin by means of mechanically operated control structures on the right bank of the Mississippi. Enlargement and extension of the main line levees to connect to these structures will obstruct an existing navigation route and therefore require provision of a lock in a channel to connect the Mississippi and Old Rivers. Construction of the control structures has been commenced but the lock is not scheduled to be started for several years. The Congressional authorization provided that the Chief of Engineers, Department of the Army, would determine the type and dimensions of the lock. As of 1956, studies of these features are in progress.

In 1946, Congress modified the project, "Red River below Fulton, Arkansas" by authorizing construction of a navigation channel from the Mississippi River to Shreveport, Louisiana to be known as the Overton-Red River Waterway. This project will follow Old and Red Rivers for about 31 miles, thence the route will be in a land cut following generally along the right bank of the Red River to a turning basin on Bayou Pierre at Shreveport, a distance of about 205 miles. The project as authorized included nine masonry locks having lengths of 650 feet, widths of 55 feet, and depths of 12 feet on the sills. In 1956, planning of the project is in progress.

At the request of Congress in 1943, a report was made by the Corps of Engineers on provision of a seaway canal 36 feet deep and 500 feet wide extending 70 miles as a land and water cut from deep water in the Gulf of Mexico to a turning basin south of Micheaud and thence by means of a connecting channel of the same dimensions to the existing inner harbor canal in New Orleans. The report contemplated the provision at a later date, when economically justified, of an additional lock near Meraux, with connecting channels and appurtenances between the turning basin and the Mississippi River. The tentative lock dimensions mentioned in the report are a length of 760 feet, a width of 110 feet, and a depth of 40 feet on the sills. As of 1956, the project has not received Congressional authorization but there has been a renewed interest shown in construction of the works.

ACKNOWLEDGMENT

Design and construction of the Empire and Ostrica Locks were performed by the Department of Public Works of the State of Louisiana and data thereon have been extracted from biennial reports of the Department. The other navigation improvements described herein are responsibilities of the Corps of Engineers, U. S. Army, and have been carried out under the supervision of the Chief of Engineers, Department of the Army, under the direction of the Division Engineer, Lower Mississippi Valley Division, Corps of Engineers.

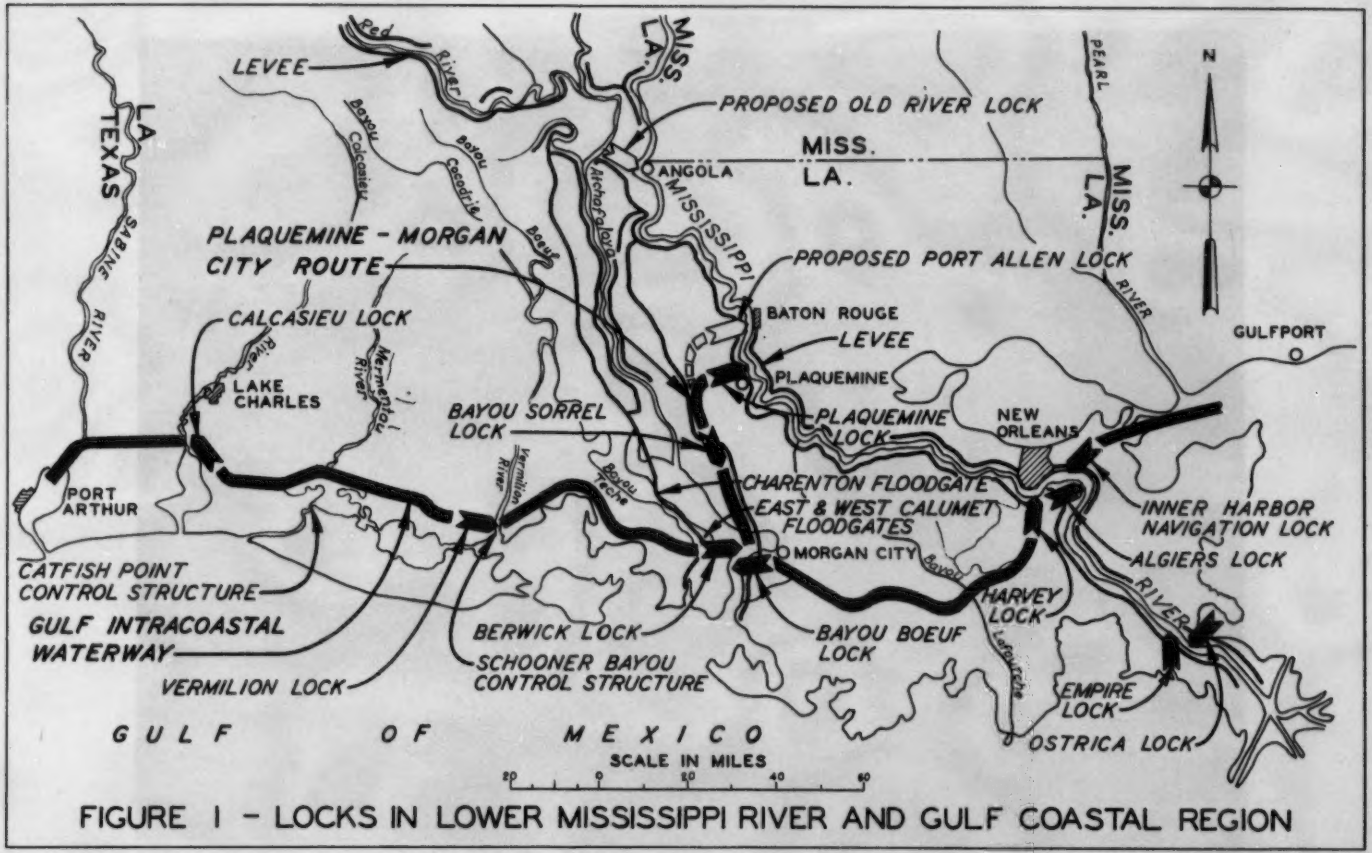


FIGURE 1 - LOCKS IN LOWER MISSISSIPPI RIVER AND GULF COASTAL REGION

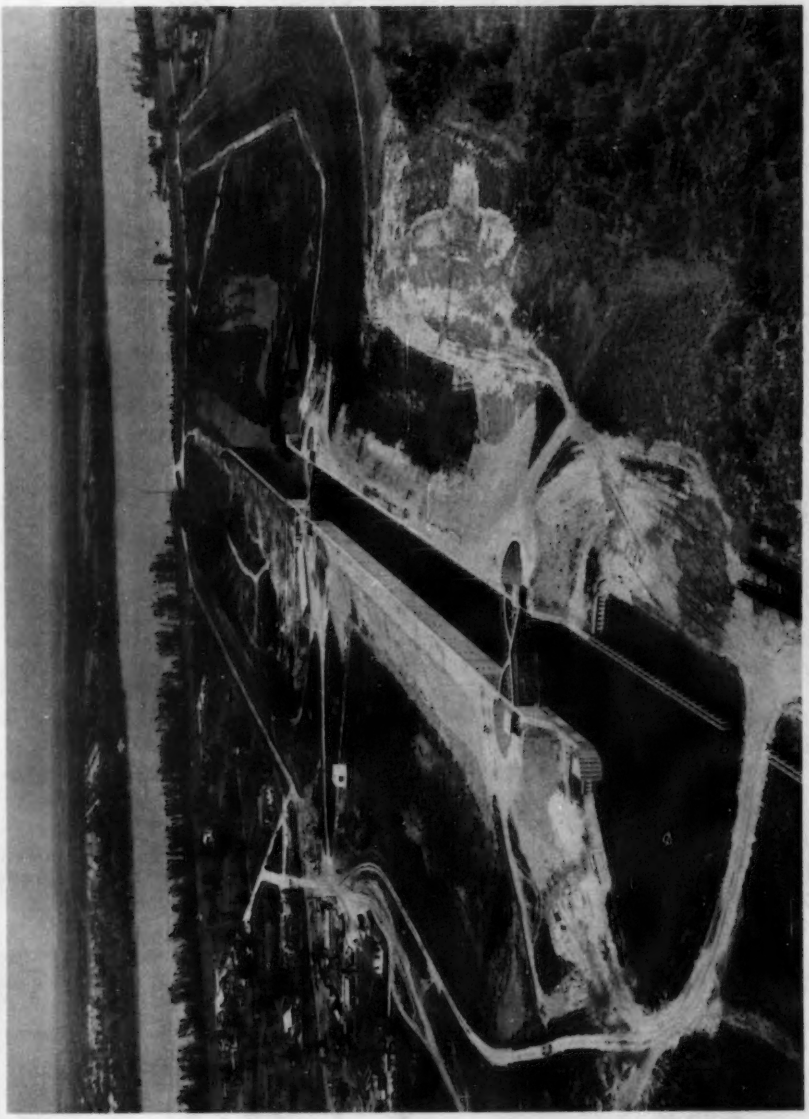


FIG. 2. ALGIERS LOCK

FIG. 3. BAYOU SORREL LOCK

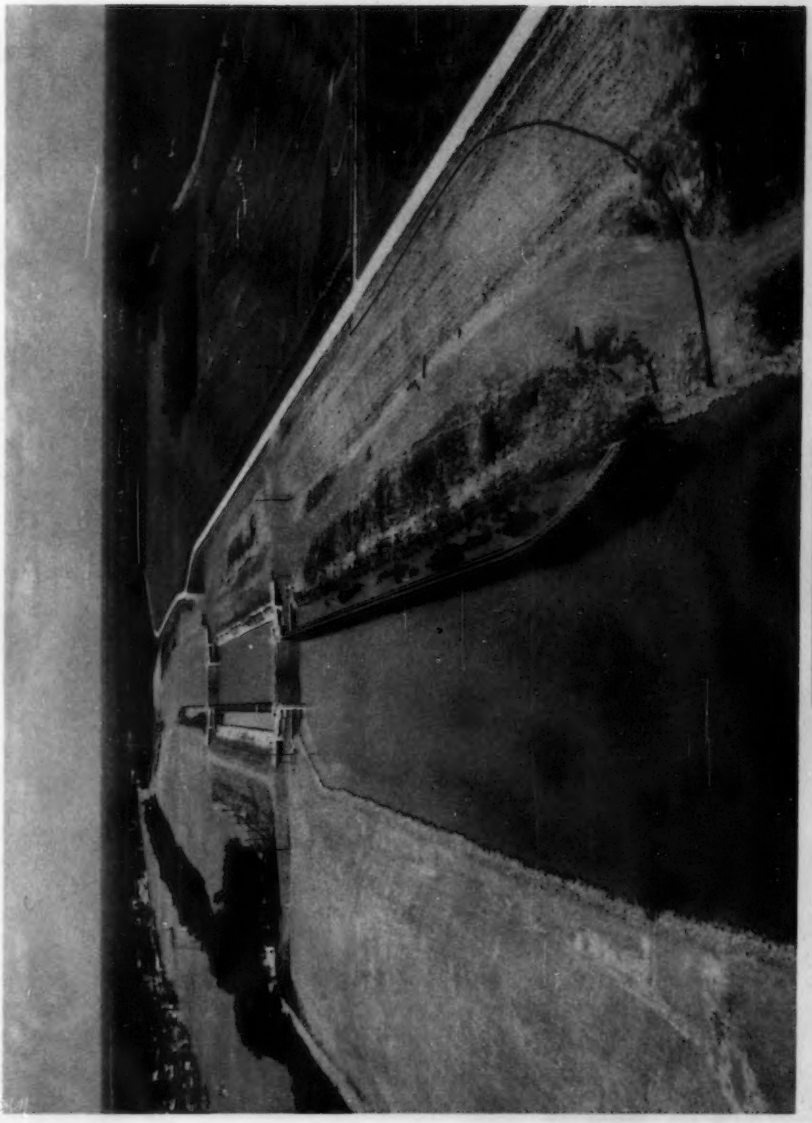


FIG. 4. BAYOU BOEUF LOCK

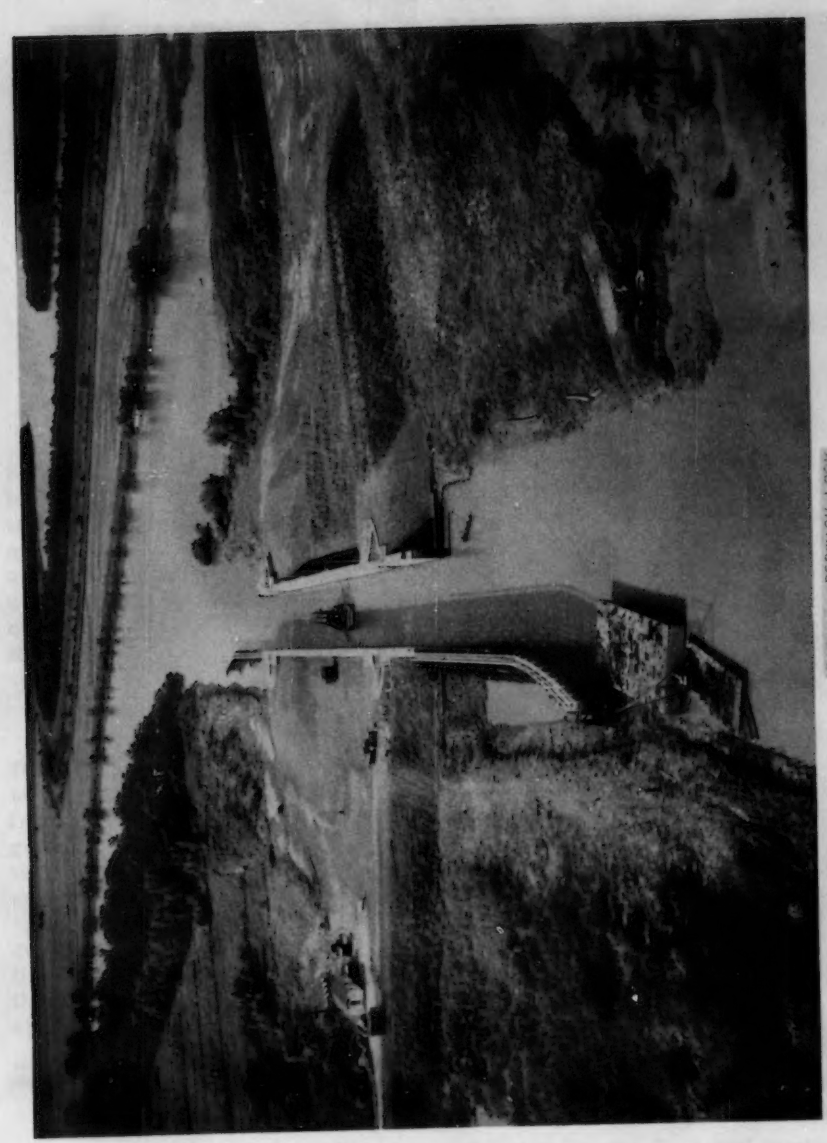


FIG. 5. BERWICK LOCK

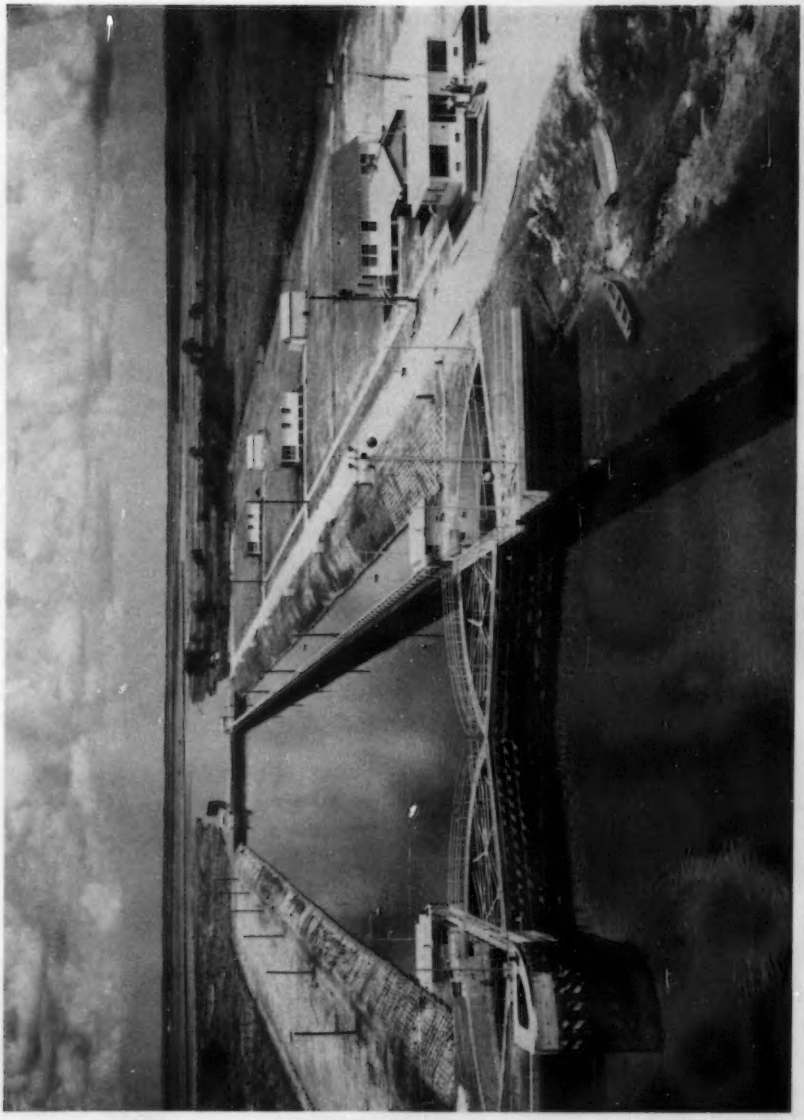


FIG. 6. CALCASIEU LOCK

Paper 975 WW 3

JOURNAL
WATERWAYS AND HARBORS DIVISION
Proceedings of the American Society of Civil Engineers

CONTROL OF RIO GRANDE BY UNITED STATES AND MEXICO

J. C. Bustamante[1] and J. F. Friedkin,[2] Members, ASCE
(Proc. Paper 975)

SYNOPSIS

For 1240 miles the Rio Grande forms the boundary between the United States and Mexico. Since 1948 when the boundary was established by treaty between the two countries, the problems of channel re-alignment, flood control, irrigation and power development have been dealt with successfully and amicably by a joint commission presently known as the "International Boundary and Water Commission, United States and Mexico." The history and accomplishments of the Commission are detailed by the authors who are principal engineers of the Mexican and United States sections, respectively, of this body.

Since the Treaty of Peace, Friendship and Limits concluded February 2, 1948, which established the middle of the Rio Grande as the portion of the boundary between the United States of America and the United Mexican States, from El Paso, Texas - Juarez, Chihuahua to the Gulf of Mexico, the two governments have jointly accomplished a record long and unique among nations for practical solution of the mutual engineering problems of the two countries as they have arisen on this river.

In its 1240-mile course, which forms the boundary, the Rio Grande meanders through a series of alluviated valleys separated by canyon reaches, and finally the rich deltaic lands of the Lower Rio Grande Valley. At the time of the 1848 Treaty, the river, then in practically its virgin state, probably discharged an average of about eight million acre-feet annually to the Gulf.

Note: Discussion open until October 1, 1956. Paper 975 is part of the copyrighted Journal of the Waterways and Harbors Division of the American Society of Civil Engineers, Vol. 82, No. WW 3, May, 1956.

1. Prin. Engr., International Boundary and Water Comm., U. S. and Mexico, Mexican Section, Ciudad Juarez, Chihuahua, Mex.
2. Prin. Engr., International Boundary and Water Comm., U. S. and Mexico, U. S. Section, El Paso, Tex.

Flows varied from almost annual lows of a few hundred second-feet during the spring seasons, to relatively frequent floods during the summer and fall seasons which spilled out of the channel, to rare floods in the same seasons of more than one million second-feet which covered the alluvial valleys and the delta lands. The river carried an average of more than twenty million tons of silt annually at the head of its delta. Its meanders were continuously modified by slow erosion of the channel banks, and at times of flood avulsive changes occurred in the river's course.

In 1848 there were along the river only sparse settlements, some scattered ranches, and a few isolated tracts under irrigation. There was no concern as to changes in the river's meanders, nor as to floods overflowing its banks. There was, of course, no need for river regulation works nor determination of the respective national rights to the waters of the river. The principal concern at that time was use of the river for navigation; and to preserve this use the Treaty provided that the river throughout its boundary course shall be free and common to the vessels and citizens of both countries, and that neither country shall, without the consent of the other, construct any works which may impede or interrupt navigation.

The first joint engineering by the two Governments was the survey authorized by the 1848 Treaty, "In order to designate the boundary line with due precision, upon authoritative maps, and to establish upon the ground landmarks which shall show the limits of both republics." This survey, conducted under the direction of a United States Commissioner and a Mexican Commissioner, appointed by their respective Governments, was completed in 1852. In the performance of this work the survey parties were escorted by troops from the armies of the two nations for protection against hostile Indian bands along the border.

The 1884 Treaty

In the years following, settlers of the two countries began to occupy lands adjoining the Rio Grande in increasing numbers. Serious questions began to arise as to the national and private ownership of properties in locations where the river shifted its course. In effort to avoid difficulties in this respect, the two Governments concluded the Treaty of 1884, wherein it was agreed that the dividing line shall forever be that described in the 1848 Treaty and follow the center of the normal channel notwithstanding any alterations in the banks or in the course of the river, provided that such alterations be effected by natural causes through the slow and gradual erosion and deposit of alluvium and not by the abandonment of an existing river bed and the opening of a new one. The 1884 Treaty provided further that any other change wrought by the force of the current, whether by the cutting of a new bed, or when there is more than one channel by the deepening of another than that which marked the boundary at the time of the survey, shall produce no change in the dividing line as fixed by the boundary surveys in 1852.

Establishment of the International Boundary Commission

To facilitate the carrying out of the principles contained in the Treaty of 1884, the two Governments by Treaty in 1889 created the International Boundary Commission; and entrusted the Commission with exclusive jurisdiction over "All differences or questions that may arise on that portion of the

frontier between the United States of America and the United States of Mexico where the Rio Grande and Colorado River form the boundary." The Commission as then established, consisting of a United States Commissioner appointed by the President of the United States of America, and a Mexican Commissioner appointed by the President of the United States of Mexico, with their respective staffs, has since been maintained in substantially its same form and has been charged by treaty obligation and by national laws in each country with increasing responsibility to cooperatively work out the common engineering problems arising along the boundary.

Treaty of 1905

As a result of its surveys relating to changes in the fluvial boundary, the Commission observed that there is a typical class of changes effected in the bed of the Rio Grande, in which, owing to slow and gradual erosion, coupled with avulsion, the river abandons its old channel, separating from it small portions of land known as "bancos" which are bounded by the old river bed, and which, according to the terms of the 1884 Treaty, remained subject to the dominion and jurisdiction of the country from which they have been separated. It was found that the application of those terms, which would result in numerous cases of one country having sovereignty over segregated portions of land on the side of the river of the other country, rendered more difficult instead of simplifying solution of controversies arising from changes in the boundary. To avoid such difficulties the two Governments by Treaty in 1905 agreed that the boundary line shall follow the center of the normal channel of the river and the dominion and jurisdiction over such portions of land as are segregated from one side of the river and attached to the other as a result of slow and gradual erosion coupled with avulsion, shall pass to the country to which the land becomes attached. The Treaty excepted from this rule portions of land segregated which have an area of over 618 acres (250 hectares) or a population of over 200 souls. Under this treaty, the Commission has settled 219 cases involving the formation of bancos on the Rio Grande, the most recent being one which formed in 1954 near Del Rio, Texas, incident to the great flood in that year.

The Treaty of 1906

By the 1880's irrigation had developed along the Rio Grande in the El Paso-Juarez Valley to the point that shortages of water in the river became serious, and as a consequence, the question of the national ownership of the waters in that area became imminent. The situation led to the first treaty between the United States and Mexico for distribution of waters of the Rio Grande between the two countries—the Treaty of 1906. Therein it is provided that after completion of Elephant Butte Dam on the Rio Grande the United States shall annually deliver to Mexico 60,000 acre-feet in the bed of the river at the heading of the main Mexican canal just above the City of Juarez; with the understanding that in case of extraordinary drought or serious accident to the irrigation system in the United States, the amount delivered to the Mexican canal shall be diminished in the same proportion as the waters delivered to lands under the irrigation system in the United States. The terms of this treaty, dividing the waters of the river above Ft. Quitman, became effective in 1916 when Elephant Butte Dam was completed, and are administered by the Commission.

Lower Rio Grande Flood Control Project

By the 1920's urban and agricultural developments along the Rio Grande in its delta, known as the Lower Rio Grande Valley, had grown to the point that floods in the river caused serious damage to properties in both countries. Solution of this flood problem required coordinated protective works in the two countries; and in 1930 the two Governments instructed their respective Boundary Commissioners to develop an international plan for flood control in the Lower Rio Grande Valley. The plan, prepared jointly by the United States and Mexican Sections of the Commission, was approved by the two Governments in 1932, authorizing the Lower Rio Grande Flood Control Project. This project extends from the town of Peñitas, Texas, to the Gulf of Mexico, a distance of about 180 river miles. Upon completion it will provide protection against river floods for the cities and communities in the valley, in both countries, aggregating a population of about 600,000, and including Brownsville, Harlingen, and McAllen, Texas, and Matamoros and Reynosa, Tamaulipas; and for the approximately one million acres of lands presently irrigated on both sides of the river in the valley.

The completed portions of the project, constructed through the United States and Mexican Sections of the Commission with each performing the work on its side of the river, include a system of levees skirting the wide meandering loops of the channel to form a river floodway, and leveed interior floodways in both countries located along natural depressions through which flood waters in excess of the capacity of the river floodway are conducted to separate outlets to the Gulf. The project when completed is designed to provide protection against a flood of 187,000 second-feed in the river at the head of the valley. In the event of such a flood flow, it is contemplated that 112,000 second-feet will be transported to the Gulf through the interior floodways in the United States, and 45,000 second-feet through the interior floodways in Mexico, leaving 30,000 second-feet in the river as it passes the border cities of Brownsville and Matamoros, where the capacity of the river is limited to that rate of flow.

The Marte R. Gomez dam constructed by Mexico on the Rio San Juan in 1943, and the Falcon dam constructed by the two Governments through the Commission in 1953, contribute to the regulation of floods in the Lower Rio Grande.

Appurtenant to the Lower Rio Grande Flood Control Project and of interest from the river regulation standpoint are the installations of woven wire mesh revetment along caving banks where developed properties are threatened with destruction or serious damage. These installations, consisting of a series of woven wire mesh fences built transverse to and sloping out from the high bank into the stream, have been generally successful in alleviating bank erosion through retarding the current and causing deposition along the caving banks.

Rio Grande Rectification Project

In the El Paso-Juarez Valley, surveys in the 1920's showed that due to the high degree of regulation of natural flow, the channel of the Rio Grande had diminished in capacity to the extent that infrequent uncontrolled floods threatened not only the irrigated lands on both sides of the river, totaling about 100,000 acres, but also the adjoining border cities of El Paso, Texas

and Juarez, Chihuahua which presently have a combined population of about 350,000. Moreover, with the deterioration of the channel each flood threatened to cause major changes in the course of the river, and hence the boundary, in this highly developed valley. Recognizing that solution to this problem required the cooperative effort of both countries, the two Governments in 1927 instructed the respective members of the International Boundary Commission to jointly develop a plan for remedial works. The recommended plan, providing for rectification of the Rio Grande through the El Pase-Juarez Valley, was approved by the two Governments and by Treaty in 1933, they resolved to construct the recommended works.

This project, constructed jointly through the United States and Mexican Sections of the Commission in the years 1934 to 1938, straightened the Rio Grande and the boundary, from its former tortuous length of 155 miles through the El Paso-Juarez valley, to a length of 88 miles. Of special interest, the straightening was effected by excavating a new channel across the bends of the old river in such manner that the area of lands cut off from the United States and attached to Mexico equalled the area of lands cut off from Mexico and attached to the United States. The rectified channel consists of a floodway about 600 feet in width bounded by levees and containing a pilot channel which carries the normal flows in the river. The design capacity of the floodway is 11,000 second-feet. To assure stabilization of this straightened section of the river boundary, it has been found necessary, as a part of maintenance, to revet with riprap about 60% of the length of the banks of the pilot channel. As a part of the rectification project flood control storage was provided on the river at Caballo dam about 110 miles above El Paso-Juarez.

Rio Grande Canalization Project

In 1936, the Congress of the United States of America authorized construction, operation and maintenance by the United States Section of the Commission of a project to canalize the 110-mile reach of the Rio Grande from Caballo dam to El Paso, in order to facilitate compliance with the Treaty of 1906, and to properly regulate and control the water supply for use in the two countries as provided by that Treaty. The construction of this project, known as the Rio Grande Canalization Project, was undertaken in 1938 and largely completed in 1943. It provides a relatively straight floodway channel varying from 400 to 1,000 feet in width, bounded by levees, which generally follows and contains the alignment of the old normal flow channel. Within the floodway, the old channel has been straightened and narrowed to some extent by excavating a new channel but for the most part by the river itself, guided and abetted by the construction of rock-armored earth jetties. When the channel reaches the desired alignment and width, its banks are revetted with riprap. The normal flow channel which carries the irrigation waters released from storage in Elephant Butte and Caballo reservoirs for lands in the United States and the 1906 Treaty deliveries to Mexico, has a capacity of about 2,500 second-feet. The floodway, including the normal flow channel, has a capacity varying from 12,000 to 22,000 second-feet; the higher capacities obtain near the mouths of large tributary arroyos which are the source of the maximum floods in the area.

The 1944 Water Treaty

In the 1930's irrigation uses by the two countries along the Rio Grande below Ft. Quitman, increased to the extent that the ordinary flows in the river were insufficient during the growing seasons, while annually an average of about three million acre-feet, largely flood waters, wasted to the Gulf of Mexico. It became abundantly clear that equitable division between the two countries of the waters of the river was urgently needed, and that storage dams to regulate the flood and other flows which wasted to the Gulf, were essential to sustain existing uses and permit development of any further feasible uses. Concurrently there developed need for equitable division of the waters of the Colorado River, which forms the international boundary for a distance of about 20 miles between the States of Arizona and Baja California, and of the Tijuana River which sources partly in each country, flows from Mexico into the United States near Tijuana, Baja California, and discharges into the Pacific Ocean.

Recognizing these needs and taking into account that the prior treaties regulated the waters of the Rio Grande and Colorado River for purposes of navigation only, the two Governments concluded the Water Treaty of 1944, which fixed and delineated the rights of the two countries with respect to the waters of the Rio Grande below Ft. Quitman, and of the Colorado River; provided for the storage and diversion works to obtain the most satisfactory and complete utilization thereof; and provided for studies and investigations looking to equitable division and maximum feasible utilization of the waters of the Tijuana River.

The Treaty provides that the application of its terms and the regulation and exercise of the rights and obligations which the two Governments assumed thereunder, are entrusted to the International Boundary and Water Commission, United States and Mexico (formerly the International Boundary Commission). This Commission, as re-established, consists of a United States Section and a Mexican Section, each headed by an Engineer Commissioner, with the former Section operating under the policy supervision of the Department of State of the United States and the latter under the similar supervision of the Ministry of Foreign Relations of Mexico.

Division of the Waters of the Rio Grande

Of the waters of the Rio Grande from Ft. Quitman to the Gulf of Mexico, the 1944 Treaty allotted to Mexico: all of the waters of its tributary San Juan and Alamo Rivers including the return flows from the lands irrigated from these two rivers which reach the Rio Grande below the lowermost feasible reservoir site on the main channel; two-thirds of the contributions from its tributary Conchos, San Diego, San Rodrigo, Escondido and Salado Rivers and Las Vacas Arroyo, which enter the river above the lowermost reservoir site; and one-half of all other flows in the main river below Ft. Quitman, not otherwise allotted.

The Treaty allotted to the United States: all the contributions to the Rio Grande from its tributary Devils and Pecos Rivers, Goodenough Spring, Alamito, Terlingua, San Felipe and Pinto Creeks; one-third of the contributions from the named Mexican tributaries above the lowermost reservoir site on the Rio Grande with the understanding that the amount shall average not less than 350,000 acre-feet annually; and one-half of all other flows in the

main river below Ft. Quitman not otherwise allotted.

Determination of the national ownership of waters pursuant to the 1944 Treaty allocations requires measurement and recording of the contributions to the Rio Grande, the quantities of water in the river and the diversions therefrom, in a manner mutually acceptable to both countries. To meet this requirement, the Treaty provides that the Commission shall jointly construct and operate on the Rio Grande and each Section shall construct and operate on the principal tributaries in its own country the stream gaging stations necessary for the accounting of the waters. Accordingly, the two Sections of the Commission maintain an extensive joint stream gaging program on the Rio Grande.

International Storage Dams Program

To effect maximum feasible utilization by the two countries of the waters of the Rio Grande within the specified allotments, the two Governments agreed in the 1944 Treaty to construct jointly through their respective Sections of the Commission, three major storage dams on the main channel of the river for the conservation and regulation of its flows. It is provided that the cost of construction, operation and maintenance of each of the storage dams shall be prorated between the two Governments in proportion to the capacity allotted to each country for conservation purposes in the reservoir at each dam. The Treaty further provides, subject to the approval of the two Governments, for the construction of hydroelectric plants as may be feasible at the international storage dams on the Rio Grande, the cost of which shall be divided equally between the two Governments and the energy generated assigned to each country in like proportion.

Falcon Dam and Power Plant

The Commission began construction in 1950 of the first international storage project on the Rio Grande—the Falcon Dam and Power Plant, located about 75 miles downstream from Laredo, Texas—Nuevo Laredo, Tamaulipas and about 50 miles upstream from the Lower Rio Grande Valley. The dam, completed in 1953, is a 5-mile long compacted earth and rockfill structure with a maximum height of 150 feet, and will impound waters to form a lake approximately 60 miles long and up to 11 miles wide when filled to its capacity of 4,085,000 acre-feet. This total capacity is made up of 2,100,000 acre-feet of conservation storage and 300,000 acre-feet of silt storage, both divided 58.6% to the United States, and 41.4% to Mexico, and 1,685,000 acre-feet of flood control storage in which each country has an undivided interest. The project includes two identical power plants, one on each side of the river, with a capacity of 31,500 kilowatts for generation of hydroelectric energy incident to releases of water by the two countries for domestic and irrigation purposes. (See article on construction of Falcon Dam by C. M. Ainsworth, M. ASCE, in CIVIL ENGINEERING, May 1951, p. 38.)

At the Dedication of Falcon Dam on October 19, 1953, the President of the United States of America, Dwight D. Eisenhower, stated:

> "More than a mute monument to the ingenuity of engineers, this Falcon Dam is living textimony to the understanding and the cooperation binding our two peoples."

And the President of the United Mexican States, Adolfo Ruiz Cortines, stated:

> "Falcon Dam symbolizes, in a most special way, the desires of our two countries to unite their efforts in this sphere of cooperation which their neighborhood makes imperative, to facilitate—and if possible to accelerate—the forward march of social and economic progress."

Since storage began in Falcon reservoir in August 1953, its regulation of flood waters has thrice averted serious shortages of water for the approximately 1,000,000 acres of irrigated land in both countries in the Lower Rio Grande Valley. (See article on irrigation development in Mexico by J. C. Bustamante, M. ASCE, in CIVIL ENGINEERING, May 1951, p. 42) In July 1954, the reservoir contained the maximum flood of record on the Rio Grande, which originated on the Pecos and Devils Rivers and had a peak discharge of about 1,100,000 second-feet at Del Rio, Texas-Ciudad Acuña, Coahuila, saving the Lower Rio Grande Valley from disastrous losses. Had the dam not been built, the magnitude of the flood in the Lower Valley would have far exceeded the design capacity of the flood protective levees in that area. On the United States side alone damages would have been in the order of 55 million dollars. In addition, the regulation of this flood provided a full water supply for the irrigated lands downstream in both countries during the past year. Studies indicate that the operation of Falcon Dam and reservoir will reduce the average annual quantity of water which formerly wasted to the Gulf—nearly 3,000,000 acre-feet, to about 800,000 acre-feet. Since the beginning of operation of the power plants at Falcon in October 1954, more than 260 million KWH have been generated.

Additional International Control Works Needed on the Rio Grande

Completion of the Lower Rio Grande Flood Control Project is needed to provide the high degree of flood protection warranted by the densely populated urban centers and extensive irrigated lands in both countries in the delta of the river. Its completion requires principally the construction of a gated control dam in the river channel to assure the design use of the interior floodways to transport to the Gulf flood waters in excess of the river's capacity. The two Governments have authorized and appropriated funds for such a structure, referred to as the Anzalduas Dam. Plans for this structure have been completed, agreed upon by the Commission and approved by both Governments; and construction will be undertaken through the two Sections of the Commission during the current year. The dam will be located just below the inlet to the Mission floodway in the United States and about 5 miles upstream from the border cities of Hidalgo, Texas and Reynosa, Tamaulipas. It will consist of a reinforced concrete structure containing 6 roller gates, each 75 feet long and 21 feet in height, which, with the piers, will span the channel width—525 feet. The estimated cost of the structure is about 5 million dollars which will be divided equally between the two Governments.

With continued proper maintenance of the Rio Grande Rectification Project, including stabilization of the channel banks with riprap revetment, the intensive urban and irrigation developments in both countries in the El Paso-Juarez Valley are assured of a high degree of protection against river floods as well as a stable boundary.

The limitrophe portion of the Rio Grande is still uncontrolled, however, from the lower end of the Rectification Project near Ft. Quitman to Falcon

reservoir near Laredo-Nuevo Laredo, a distance of about 800 river miles. In this uncontrolled section the most important developments in the two countries are situated in the 200-mile reach from the mouth of Devils River to Falcon reservoir, wherein are located the adjoining border cities of Del Rio-Ciudad Acuña, Eagle Pass-Piedras Negras, Laredo-Nuevo Lardeo, aggregating a population of about 300,000 and irrigated lands in the two countries totaling about 80,000 acres. This reach lying below the mouth of the Pecos and Devils Rivers, the sources of the greatest floods in the Rio Grande, is subject to such devastating damages as occurred incident to the record 1954 flood, when many lives were lost and property losses amounted to more than 20 million dollars.

Plans are currently being prepared by the Commission for control of this reach of the Rio Grande. They contemplate construction of the second major storage dam on the main channel at the dam site, referred to as the Diablo site, located about one mile below the mouth of the Devils River, under the international storage dams program authorized by the 1944 Treaty. The proposed Diablo Dam would regulate to the capacity of the channel downstream all floods of record at the dam site, including the unprecedented 1954 flood. It would, in combination with the Falcon Project, regulate flood waters to the extent of permitting almost complete utilization of the waters of the Rio Grande by the two countries. Incidental to these purposes, the Diablo Dam would permit generation of a large block of hydroelectric energy.

In the remaining 600 miles of the uncontrolled section of the Rio Grande, from Ft. Quitman to Del Rio-Ciudad Acuña, its course is through a series of canyons, including those in the Big Bend region, and intervening narrow valleys where some irrigation is practiced. The developments are not extensive except in the Presidio Valley where about 20,000 acres are under cultivation in the two countries. This area is subject to flood damages and suffers shortages of water during drought periods.

To the famed canyons of the Big Bend region, the total fall in the river's profile is over 1,600 feet, and a number of dam sites exist in the almost vertical rock walled incisions cut by the river to depth of as much as 2,000 feet. There is the potential development in this reach of a series of dams for generation of large blocks of hydroelectric energy for use in the two countries.

Direction of the International Program

The long-term and specific program adopted in treaties between the Governments of the United States and Mexico and entrusted to the International Boundary and Water Commission, for practical and mutually beneficial solution of the joint engineering problems along the 1935 miles of common boundary, including the Colorado River and the land boundary as well as the Rio Grande, is being carried on under the direction of Leland H. Hewitt, Commissioner of the United States Section, and David Herrera Jordán, Commissioner of the Mexican Section of the Commission.

JOURNAL
WATERWAYS AND HARBORS DIVISION
Proceedings of the American Society of Civil Engineers

SAND BY-PASSING AT A VIRGINIA TIDAL INLET

Thomas J. McDonald[1] and Myron A. Sturgeon,[2] M. ASCE
(Proc. Paper 976)

SYNOPSIS

Progressive beach erosion at Virginia Beach, Virginia, resulted in a critical impairment of that resort city's economy. A program of beach restoration and maintenance was undertaken in 1951, to carry out recommendations of the Corps of Engineers and U. S. Beach Erosion Board. The essential features of the program were initial restoration by direct placement of sand throughout, and continued maintenance by periodic replenishment at certain locations. The barrier effect of a nearby tidal inlet on the alongshore drift, and thus on beach erosion was studied. The necessity and feasibility of transporting a fraction of the beach material carried in the alongshore drift across the inlet barrier to nourish the downdrift beach were determined.

This paper sets out the criteria developed for the design of a sand by-passing system to accomplish this transport, and describes the system and its operation.

INTRODUCTION

In September, 1951, a large scale program of restoration and maintenance of the city's badly eroded ocean beach was undertaken at Virginia Beach, Va. The program was designed to carry out recommendations of the U. S. Beach Erosion Board and the Corps of Engineers, formulated as a result of the study begun in 1948. In brief, these recommendations contemplated initial restoration by the direct placement of sand in the problem area, and maintenance by periodic replenishment. The program is now in the maintenance stage, the initial restoration having been completed in July 1953.

The program is administered by a Commission, created by an Act of the General Assembly of Virginia, and is financed by funds made available in part by the City of Virginia Beach, and in part by the State and Federal Governments.

Note: Discussion open until October 1, 1956. Paper 976 is part of the copyrighted Journal of the Waterways and Harbors Division of the American Society of Civil Engineers, Vol. 82, No. WW 3, May, 1956.

1. Chf. Engr., Myron Sturgeon Engrs., Norfolk, Va.
2. Cons. Engr., Norfolk, Va.

Virginia Beach is an important resort community, situated on the Atlantic Ocean 5 miles south of Cape Henry. The problem area comprises the entire segment of beach within the corporate limits, approximately 3 miles in length. Just at the south corporate limit, a small headland stream, known as Owl's Creek, discharges into the ocean through Rudee Inlet. Dredging operations in the lower reaches of the Owl's Creek estuary, known as Lake Wesley and Lake Rudee, provided the material used in the initial beach restoration and in periodic maintenance to date. Fig. 1 shows some of these features.

In its natural state, the inlet interfered in some degree with the alongshore drifting of sand upon which the beach in the problem area was dependent for any measure of stability. As a result of the dredging operations, this interference was considerably increased. To compensate for it, the sand by-passing system was designed and is now in successful operation.

Tidal Inlets

In dealing with problems of beach preservation and erosion control along the Atlantic Coast from Long Island to the Carolinas, the Coastal Engineer will almost inevitably be confronted with those special problems which are created by a tidal inlet. From Cape Henlopen to Cape Fear only, there are 34 such inlets, not counting the Chesapeake Bay entrance. Fig. 2 shows 17 of them between Cape Henry and Cape Fear. Most of them exhibit the systematic characteristics of interconnections between the ocean and one or another of the great sound systems. In addition to tidal flows, they must accommodate the seaward passage of large portions of the coastal plain drainage. Tidal flows may be of the order of 100,000 to 200,000 c.f.s., as at Oregon Inlet, one of the larger of the Carolina sound inlets; or 15,000 to 25,000 c.f.s., as at Moore Inlet, one of the lesser of the same system.

In general, several such inlets provide circulation and drainage within a single sound system. The partial or complete closing of one by a great storm will alter the hydraulics of another temporarily. The natural balance of these systems and the related shore forms is sensitive to immediate forces, but, in the long range, is fairly permanent, at least in the historical context. The typical inlet, while exhibiting migratory tendencies in response to changing conditions of waves, winds, and tides, may be a relatively stable feature. As an example, surveys at Oregon Inlet from 1852 to 1917 show that it migrated southward 1.2 miles in that period of 65 years. On the other hand, when Francis Drake visited Raleigh's Roanoke Colony in 1585, his ships anchored at sea off an inlet. According to the account of one Francis Pretty, who accompanied the expedition, the Governor's Fort in Roanoke Island lay northwestward about 6 leagues from the inlet. The water distance from the site of the now restored Fort to the present day Oregon Inlet is just about 19 statute miles. If the "league" is given its probable meaning of about 3 statute miles, it seems reasonable to suppose that for 370 years at least, Oregon Inlet has been about where it is now, within a mile or two.

With few exceptions, the Virginia and Carolina inlets conform to the classical pattern of inlet origins related to the barrier beach and lagoon forming processes.

Rudee Inlet appears to be one of the notable exceptions.

Rudee Inlet

Rudee Inlet is probably the smallest of the Atlantic Coast Inlets. It is not related to any existing sound system, being merely the mouth of the small estuary of Owl's Creek. The total area of the estuary is about 200 acres and most of this is tidal marsh. The Owl's Creek watershed is about 4 square miles. Within the normal range of tides, (3.0 feet), and even in periods of heavy rainfall, the ratio of sea water to fresh water in the tidal prism on the ebb tide will be not less than 4. When the tidal range exceeds 3 feet the ratio increases sharply, and when a tidal rise of 6 feet above Mean Low Water is reached, as it is occasionally, and the run-off is low, the fresh water component of the following ebb tide is virtually eliminated. Thus, it is the tidal currents which preserve the inlet, and which, under the influence of the alongshore current, regulated its migration in the natural state.

The province in which the inlet and the problem area are situated is influenced by the Chesapeake Bay entrance. Nearly always, a northward moving inshore current is observed, counter to the generally prevailing southward current along the Virginia and Carolina coasts, and the alongshore drift of beach material is dominantly northward. Thus, Rudee Inlet is usually updrift of Virginia Beach, and the latter is sensitive to the sand barrier effect of the inlet.

Estimates based on surveys from 1948 to 1953 indicate that the transport of beach material out of the problem area, mostly by alongshore drifting, is between 120,000 and 145,000 cubic yards per year.

During the same period, surveys and observation indicated that quantities of 50,000 to 95,000 cubic yards per year passed the inlet and entered the problem area. The resulting deficiencies of 50,000 to 70,000 cubic yards per year led to the progressive deterioration of the beach prior to the initial restoration.

This is not to say, however, that the barrier effect of the inlet in its natural state created the whole deficiency. For long periods each year, particularly in Summer, tidal flows in the inlet are so small as to have only a minor effect on alongshore sand drift. Up to now, no means have been found to measure the quantitative barrier effect with much certainty. Conclusive evidence of this effect is seen, however, in the condition of the downdrift beach.

Dredging operations in the estuary inside the inlet during the initial restoration increased tidal flows substantially in the normal tide range. For conditions of extreme high tide, usually associated with a northeast wind set-up, the increase in tidal flows resulting from the dredging operation was not significant. Fig. 3 shows the relationships of the tidal prism before and after the dredging operation, the tidal prism being computed as the intertidal volume in the estuary. Stream gauging in the inlet has on occasion given conformable results. In general, however, stream gauging provides less acceptable data for this case than intertidal volume measurement, the inlet cross-section being extremely sensitive to flow demands and varying from day to day. Fig. 4 shows such variation. On the other hand, the estuary is so small as to permit volumetric measurements and computations without excessive labor.

Immediately following the initial dredging operation the inlet was stabilized by means of two short stone jetties and connected bulkheads. This construction was for the purposes of preventing destructive migration of the inlet, facilitating future dredging operations for beach maintenance, and ultimately providing collateral benefits by permitting the creation of a harbor for small boats.

The By-Passing System

Having established the barrier effect of the inlet in its natural state; having projected a dredging operation which would occasionally increase tidal flows in the inlet and thereby increase this effect; and having planned the construction of two short jetties to arrest inlet migration and to facilitate the entrance of dredging equipment from the ocean, thereby further tending to increase the barrier effect, it became necessary to consider the economics of transporting intercepted drift material across the inlet barrier.

The processes involved in such transport have become fairly well established in this country and elsewhere, and the term "by-passing" has come to have a general definitive meaning when associated with alongshore drifting of beach material at tidal inlets. Most generally the by-passing is accomplished by means of hydraulic dredging equipment, either of the conventional floating type or of a specially designed, shore-mounted type. The latter may conveniently be referred to as the fixed dredging plant.

Factors determining the selection of the shore-mounted type, for this case, were:

a) The conditions of exposure, which do not favor the operation of floating equipment in the near-shore area.
b) The small quantities of material in the annual drift, which preclude the efficient use of large floating equipment.
c) The variable rate of drifting, as a result of which long periods of equipment idleness may be expected to occur.

Prototypes of the fixed dredging plant are found at South Lake Worth Inlet, Florida; at Salina Cruz, Mexico; and at Durban, South Africa. The functional demands on these plants and their performance as they have been related to the writers, and on the Rudee Inlet plant as developed by the writers, are summarized as follows:

Plant	Primary Function	Secondary Function	Annual Drift Cubic Yards	Annual By-Passing Cubic Yards
South Lake Worth Inlet	Maintenance of Downdrift Beach	Maintenance of Inlet Channel	200,000	80,000
Salina Cruz	Maintenance of Harbor Entrance	-----	1,046,000	Unknown
Durban	Maintenance of Harbor Entrance	-----	500,000	200,000
Rudee Inlet	Maintenance of Downdrift Beach	Maintenance of Inlet Channel	70,000	95,000 (Design)

It will be seen that, in every case except that of Rudee Inlet, the alongshore drift is abundant, and the plants have to handle only a portion of such drift.

At Rudee Inlet, the average annual drift quantity, for design purposes, was taken as 95,000 c.y., thus establishing a relatively small facility. Observation had shown that, not considering storm periods, a wide diversity of drift rate would be found; and that an appropriate diversity factor for most conditions would be 4. It was concluded that no attempt should be made to deal with the very large rates of drift which sometimes accompany great storms, because of the disproportionately large capital outlay which would be required; because the plant would probably be inoperative in such storms; and finally, because in such storms the direction of sand movements might very well be reversed.

With a diversity factor of 4 the maximum daily quantity for design is 1,040 c.y. Consistent with the small scale of the entire problem, it was desired to limit the operating force to about three men, and to operate a single 8-hour shift so far as possible. It was clear that the highest rates of drift would not prevail for more than a few days at a time, and that continuous operation could be maintained for these periods, if necessary, with the small force.

Accordingly, the pump capacity was fixed on the basis of the following factors:

a) The maximum daily drift to the accretion area within which the dredge is operable would be 1,040 c.y.
b) All of this drift might be intercepted and handled by the by-passing plant, under certain conditions.
c) The maximum effective operating time per day for short periods would be 20 hours.

From these considerations a pump capacity of 50 c.y. of solids per hour was taken.

Fairly comprehensive studies of beach behavior in the downdrift segment, a detailed discussion of which is not within the scope of this paper, had shown the existence of an area of especially severe erosion, centered 4,000 feet northward (downdrift) from Rudee Inlet. Beach material deposited in this area is eroded at an abnormally high rate, and transported downdrift, where it serves to maintain a condition of general beach stability. This is a feeder area, in which periodic replenishment is made. On the other hand, from this feeder area to Rudee Inlet, the beach depends for stability upon sand drift passing the inlet from the South. This drift is quantitatively adequate to satisfy the requirements of the beach for about 2,000 feet downdrift from the inlet. It is not adequate to satisfy the requirements of the segment characterized as the feeder area. This area must be replenished periodically from sources outside the beach zone. Therefore, no reason was seen for extending the discharge line from the by-passing plant to it. At the same time, it was not feasible to discharge material in too close proximity to the inlet, because occasional reversals of drift could transport recently deposited material southward into the inlet, thus defeating the purpose of the operation. Moreover, certain limitations were imposed by the seasonal uses and occupancy of various segments of the beach. All these considerations favored discharging from the by-passing plant in a segment between 1,000 and 2,000 feet north of the inlet. 2,000 feet was taken as the design length of the discharge line.

Anticipating the occasional necessity of handling fairly coarse sand, having a median grain size up to 0.5 mm, a relatively high velocity flow in the discharge line was desired. Modern practice in the selection of hydraulic dredging equipment favors velocities of about 13 feet per second. Increased efficiency will be found to compensate for increased wear in discharge piping.

Head losses in discharge pipes of 6", 8", and 10" diameters were plotted against discharges expressed as cubic yards of solids per hour. The 8" pipe fits comfortably the conditions of velocity 10 to 15 feet per second, percent solids 5 to 15, and rate of solids discharge 40 to 60 cubic yards per hour. The total head loss through the 2,000 feet of discharge will range from 100 to 200 feet depending upon the character of the material being handled at any time. The pump selected for this service was an 8" heavy duty dredging pump capable of discharging 1880 GPM against a TDH of 193 feet. For the design conditions, the efficiency of this pump is a little low, and a 6 inch pump might be satisfactory for the service. For this case, however, some sacrifice of efficiency was considered to be justified by a little reserve of pump capacity, and by a pragmatic prejudice against too small pumps for dredging.

In the selection of power sources for the main dredging pump and auxiliary equipment, only the most cursory consideration was given to electric motors. It was considered mandatory that the entire facility be made as nearly invulnerable to storm damage as could be. Accordingly, diesel power was selected for the main engine and gasolene or diesel for the auxiliary engines, the option being given to the suppliers.

The equipment components were procured following competitive bidding on a performance specification. Thus, the details of equipment design and arrangement were left, within fairly narrow limits, to the suppliers. This method of procurement of specialized equipment is considered altogether satisfactory.

The general arrangement of the equipment as finally evolved is shown by Fig. 5. Housings to protect the equipment and the operators have since been added. These were not provided in the original design because it was considered that they could be more effectively contrived after a period of operation.

The arrangement shown differs from any of the prototypes previously mentioned. The factors governing this arrangement were as follows:

a) Private ownership of property abutting the updrift beach (to the South) would not admit of any operation which might result in an excessive drawdown of sand in the beach zone. This consideration precluded the development of one of the preliminary schemes, in which the dredge operated on a modified gantry along the south side of the south jetty, moving inward and outward through a range of about 100 feet and excavating a trench through the area of accretion, normal to the beach. This arrangement would evidently have been somewhat similar to that employed at Durban.

b) The construction of a cellular structure at or near the end of the south jetty, with a suction head or intake operable in the area of accretion, would create such scouring currents as to impede the accretion of sand and thereby defeat the purpose of the facility. Such a scheme, which was closely examined, would have been similar to that employed with considerable success at South Lake Worth Inlet.

c) Occasional reversals of drift were expected to create a secondary area of accretion just inside the head of the jetty, which would impede possible future navigation and which would, in any case, represent a restorable loss of beach material from the problem area. For this reason, the dredging should, if possible, have access to this secondary accretion area.

d) Finally, the intake head should have the widest possible range consistent with the economics of the project, and the permissible size of the facility.

The adopted arrangement seems to satisfy all these requirements. The dredging area is far enough offshore to remove the possibility of damage to private property, while still within the normal area of accretion. The open platform on which the suction swing machinery is mounted permits accretion with a minimum of local current interference. The suction head can be operated directly in the secondary area of accretion, and can operate on a 46 foot radius thru 192 degrees of arc; and through a vertical range of 10 feet or more.

The crest of the south jetty is at elevation 8.0 above M.S.L. In storm periods waves break over the jetty head. Very little risk of direct wave damage to the pumping and swinging machinery was anticipated, but it was considered advisable to avert, so far as possible, the effects of salt water on the machinery. For this reason the main pump and engine and the service pump and engine are mounted on a carriage which operates over rails along the top of the jetty. In storm periods the suction line is disconnected and the carriage moved to the shore.

The bottom material underlying the accretion area is a colloidal mud, very cohesive, and quite resistant to current scour. This bottom is low enough so that no particular need exists for excavating it, and no cutting mechanism is required. The sand deposited on this bottom by alongshore drift and wave action varies according to prevailing conditions from median grain sizes of 0.2 mm to 0.5 mm. It was considered that such material would not require mechanical agitation and it was desired to eliminate the costly and troublesome cutterhead mechanism. Accordingly, provision was made for a high velocity twin water jet to be discharged at the suction intake. The jet pumping equipment, selected empirically, discharges 300 g.p.m. against a total dynamic head of 300 feet. The same pump is connected to the main bearing water seal of the dredging pump.

The entire facility, while especially designed for a unique service, was made an uncomplicated as possible in the interest of simplified operation and easy maintenance. All pumps and engines are of widely used models, for which replacement parts are readily procured. Wire ropes, sheaves, and other items of the rigging are all readily available. The structural elements are generally of rolled sections. The main pivot assembly is built up of structural shapes, no special castings or forgings being used. The facility can easily be maintained and entirely rebuilt by the operating personnel.

The suction piping is 10" extra heavy steel. Flexibility for horizontal and vertical movement is provided by insert sleeves of heavy-duty rubber dredging hose. The discharge line is 8" spiral welded steel dredging pipe having a wall thickness of 10 gauge, except for the submerged section of the line, crossing the inlet. In this location 8" class 250 cast iron with standardized mechanical joints was used.

The placing of the submerged section created the only construction problem of any consequence. In order to permit future maintenance dredging by floating equipment in the inlet, it was required that the submerged line be 10 feet below M.S.L. At the time the line was installed, the average elevation of the bottom was - 2 M. S. L. Thus a trench 8 feet deep was required across the inlet. This could not be excavated by means of any dragline or similar equipment available, because of the rapidity with which any excavation would fill up. The operation required that the entire trench be excavated; that the excavating equipment be removed from the inlet; and that the pre-assembled section of pipe line be floated into position and lowered into the trench, all within a period of approximately 8 hours, depending upon the conditions of wind and tide.

It was possible in this case to utilize a small hydraulic dredge which was working inside the inlet at the time the submerged line was to be installed. This machine was able to excavate the trench, over-dredging somewhat to allow for the filling up which would occur. The cast iron pipe had been pre-assembled on the beach inside the inlet, pontoon floats had been attached by manilla rope lashings, and the entire section, 162 feet in length, had been put afloat. As soon as the trench excavation was completed, the section was floated into place, and the lashings were cut away successively from end to end. The operation was carried out without mishap.

The standardized mechanical joint pipe was selected because time had become a factor of some consequence, and the anticipated delivery time of any ball and socket type of joint was not satisfactory. The handling and placing a pre-assembled section would obviously involve very wide deflections, and the joints would obviously be required to resist longitudinal tension. Standardized mechanical joints afford practically no resistance to tension. To meet this difficulty, one bolt was left out of each joint, and a continuous length of 5/8" ⌀ wire rope was threaded through the empty holes from end to end, and secured at each end. This arrangement permitted a sufficient amplitude of deflection in any plane, while at the same time affording such longitudinal restraint that no separation could occur at any joint. These joints, and the method of assembly, have proven altogether satisfactory and economical.

In most other respects the details of the facility conform to those of the usual dredging plant of comparable capacity. The suction lift is, of course, substantially greater than in a floating dredge. The most effective operating range is found with 18 to 22 inches of vacuum on the suction side.

Operation and Results

The total period of operational experience with the facility is a little over 10 months. Thus, conclusive data as to its functional success are not yet available. It is clear, however, that the assumed limitations imposed on its performance by variable drift rates were valid. The plant has been idle for several periods when no accretion took place, and has, on the other hand, been able to operate 16 to 20 hours per day for periods when drift and accretion were abundant.

The total quantity of sand by-passed in the ten-month period was 28,000 cubic yards. This represents about fifty percent of the estimated quantity of northward drift during that period. Several reasons for this limited production are perfectly clear. During the first four months of operation only two

men were employed, and these worked a regular 8-hour shift Monday to Friday inclusive. Thus, it was impossible to take full advantage of favorable conditions. During August, September, and October, additional men were employed, and production was greatly increased. The average effective pumping time per week during the first 4 months was only a little over 10 hours. In September, on the other hand, it was almost 30 hours. Production varies directly as the effective pumping time, of course.

Despite the present insufficiency of survey data to show the measured effects on the downdrift beach and on the inlet channel, the visible results are just as expected.

A program of continuing record surveys in the entire problem area has been underway since 1952 and is to be continued indefinitely.

ACKNOWLEDGMENTS

The writers are indebted to Mr. R. O. Eaton and other members of the staff of the U. S. Beach Erosion Board and the Corps of Engineers for their assistance in the assembling of published and unpublished data bearing on this project.

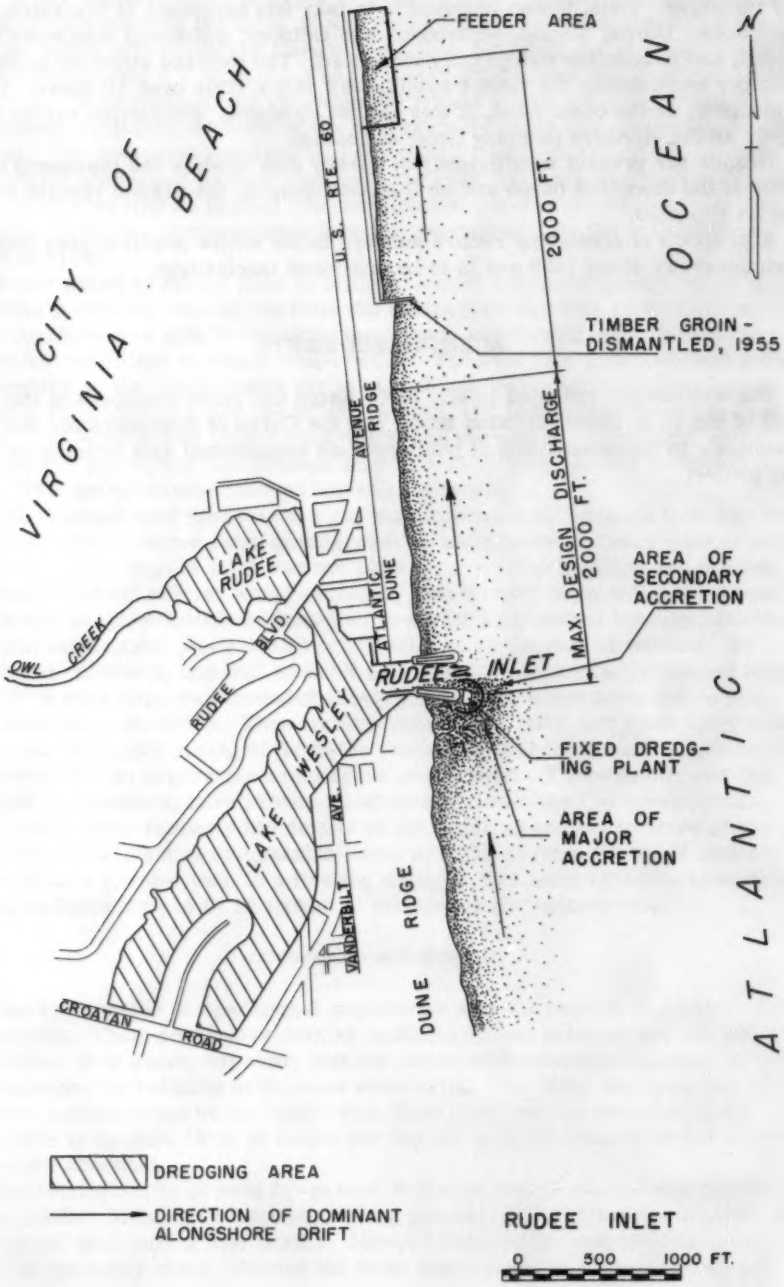

FEEDER AREA
N
CITY OF VIRGINIA BEACH
OCEAN
U.S. RTE. 60
2000 FT.
TIMBER GROIN - DISMANTLED, 1955
RIDGE AVENUE
MAX. DESIGN DISCHARGE 2000 FT.
AREA OF SECONDARY ACCRETION
LAKE RUDEE
ATLANTIC DUNE
OWL CREEK
BLVD.
RUDEE INLET
RUDEE
LAKE WESLEY
AVE.
FIXED DREDGING PLANT
AREA OF MAJOR ACCRETION
VANDERBILT
DUNE RIDGE
CROATAN ROAD
ATLANTIC
DREDGING AREA
DIRECTION OF DOMINANT ALONGSHORE DRIFT
RUDEE INLET
0 500 1000 FT.

FIG. 1

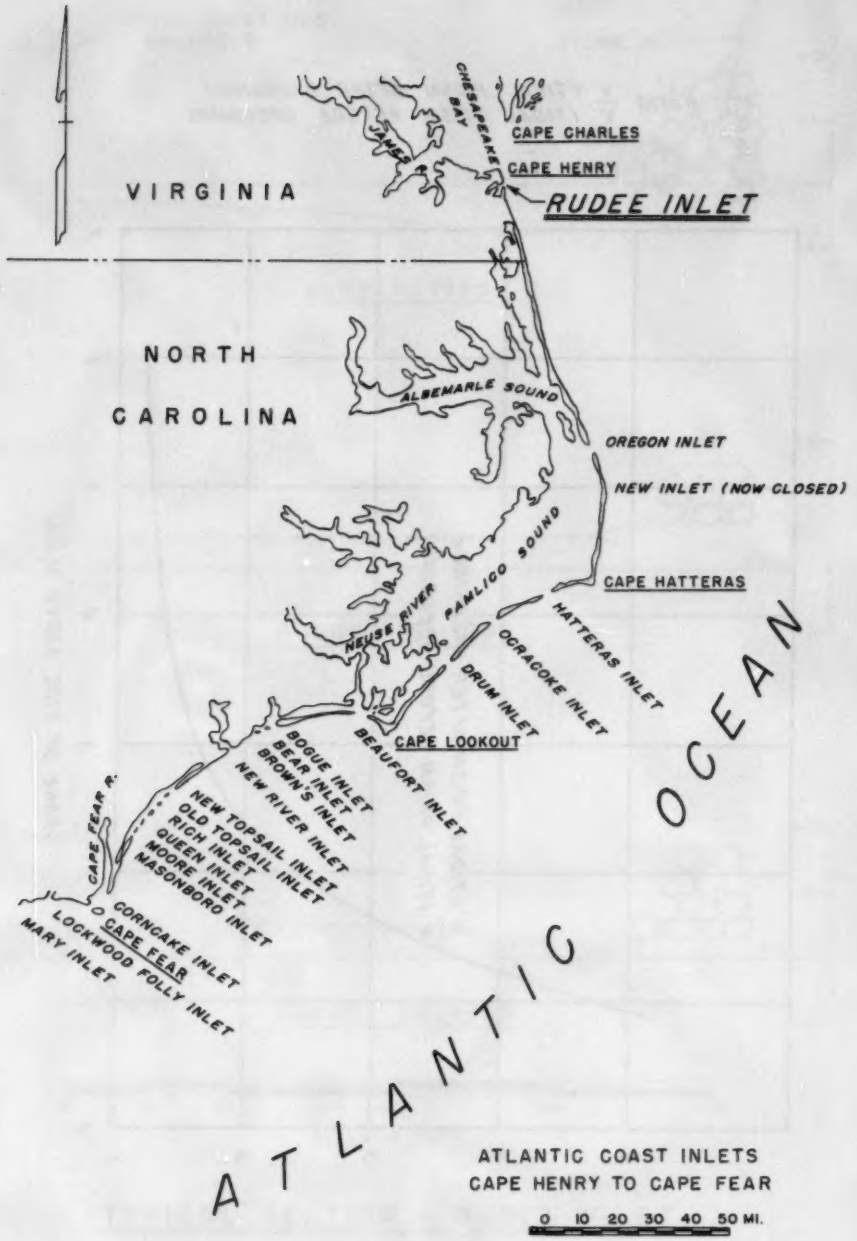

VIRGINIA
NORTH
CAROLINA
CHESAPEAKE BAY
JAMES
CAPE CHARLES
CAPE HENRY
RUDEE INLET
ALBEMARLE SOUND
OREGON INLET
NEW INLET (NOW CLOSED)
PAMLICO SOUND
CAPE HATTERAS
NEUSE RIVER
HATTERAS INLET
OCRACOKE INLET
DRUM INLET
CAPE LOOKOUT
BEAUFORT INLET
BOGUE INLET
BEAR INLET
BROWN'S INLET
NEW RIVER INLET
NEW TOPSAIL INLET
OLD TOPSAIL INLET
RICH INLET
QUEEN INLET
MOORE INLET
MASONBORO INLET
CAPE FEAR R.
CORNCAKE INLET
CAPE FEAR
LOCKWOOD FOLLY INLET
MARY INLET
ATLANTIC OCEAN
ATLANTIC COAST INLETS
CAPE HENRY TO CAPE FEAR
0 10 20 30 40 50 MI.

FIG. 2

RATIO $\frac{V'}{V}$ (TIDAL PRISM AFTER DREDGING) / (TIDAL PRISM BEFORE DREDGING)

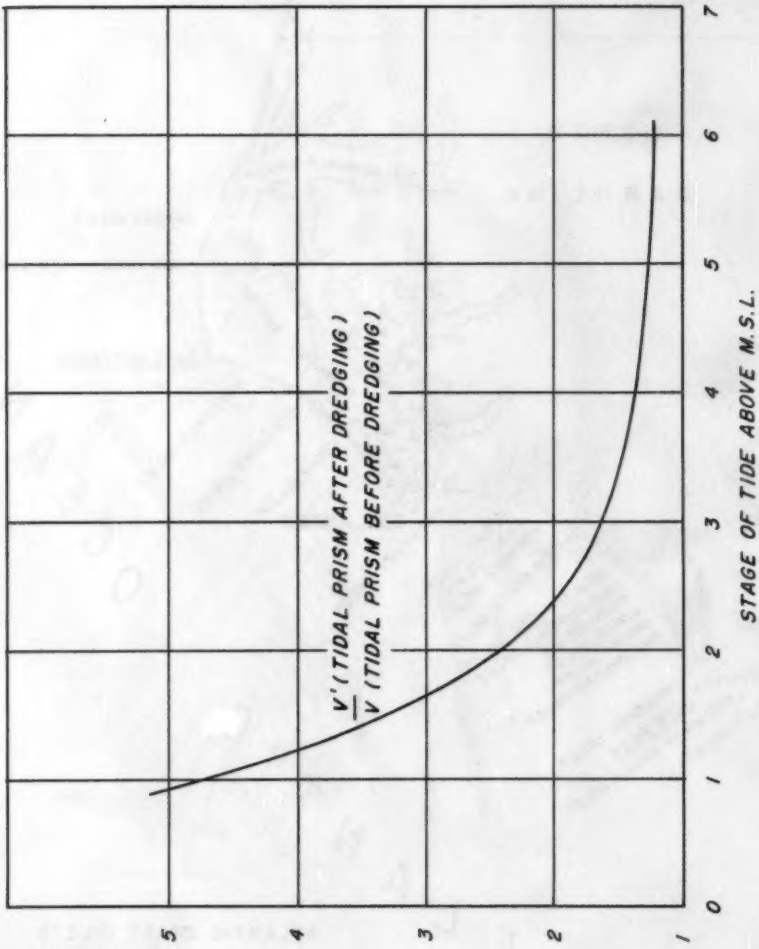
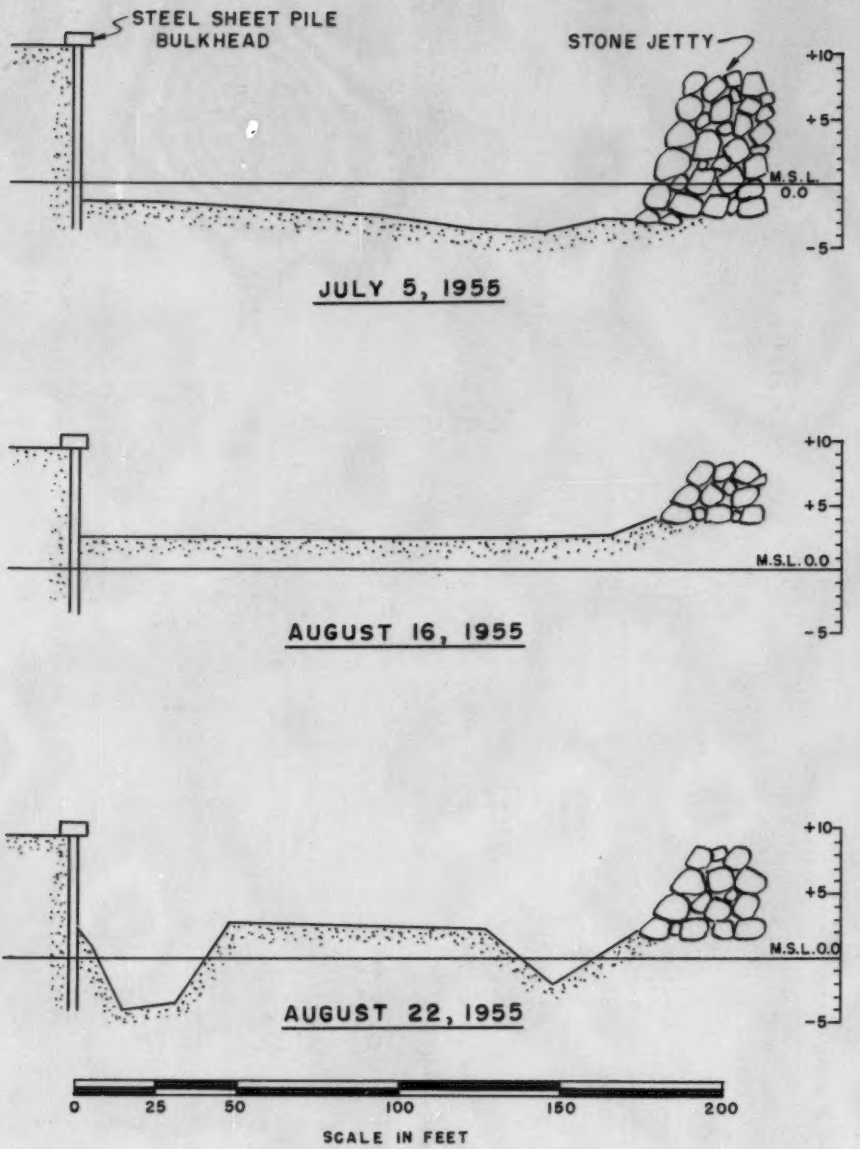


FIG. 3



TYPICAL SECTION - RUDEE INLET

FIG. 4

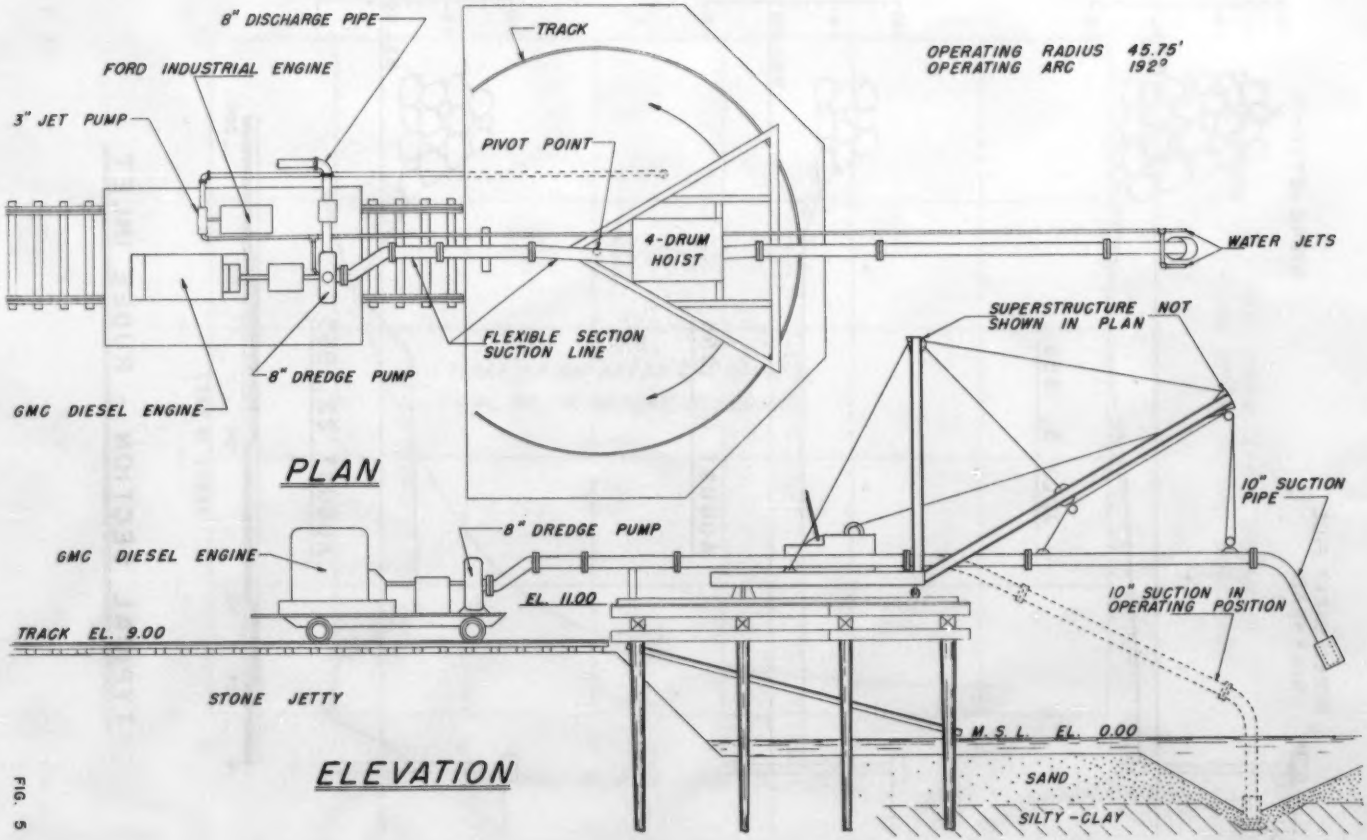

8" DISCHARGE PIPE
FORD INDUSTRIAL ENGINE
3" JET PUMP
TRACK
PIVOT POINT
OPERATING RADIUS 45.75'
OPERATING ARC 192°
4-DRUM HOIST
WATER JETS
FLEXIBLE SECTION SUCTION LINE
8" DREDGE PUMP
GMC DIESEL ENGINE
PLAN
SUPERSTRUCTURE NOT SHOWN IN PLAN
10" SUCTION PIPE
8" DREDGE PUMP
GMC DIESEL ENGINE
EL. 11.00
10" SUCTION IN OPERATING POSITION
TRACK EL. 9.00
STONE JETTY
M.S.L. EL. 0.00
SAND
SILTY-CLAY
ELEVATION

FIG. 5

TABLE OF CONTENTS

The technical papers published in each Proceedings are identified by number below. Technical division sponsorship is indicated by an abbreviation at the end of each Paper Number. The symbols referring to: Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways and Harbors (WW) divisions. Papers sponsored by the Board of Direction are identified by the symbol (BD). For the most economical order coupons, refer to the special issue of "Civil Engineering" for September, with Volume 83 (January 1956) papers will be published in Journals of the various Technical Divisions. To locate papers in the Journals, the symbols after the paper numbers are followed by a numeral designating the issue of a particular Journal in which the paper appeared. For example, Paper 861 is identified as 861 (SM1) which indicates that the paper is contained in Issue 1 of the Journal of the Soil Mechanics and Foundations Division.

VOLUME 81 (1955)

MAY: 679(ST), 680(ST), 681(ST), 682(ST)[c], 683(ST), 684(ST), 685(SA), 686(SA), 687(IA), 688(SA), 689(SA)[c], 690(EM), 691(EM), 692(EM), 693(EM), 694(EM), 695(EM), 696(PO), 697(HO), 698(SA), 699(PO)[c], 700(PO), 701(ST)[c].

JUNE: 702(HW), 703(HW), 704(HW)[c], 705(IR), 706(IR), 707(IR), 708(IR), 709(HY)[c], 710(CP), 711(CP), 712(CP), 713(CP)[c], 714(HY), 715(HY), 716(HY), 717(HY), 718(SM)[c], 719(HY)[c], 720(AT), 721(AT), 722(SU), 723(WW), 724(WW), 725(WW), 726(WW)[c], 727(WW), 728(IR), 729(IR), 730(SU)[c], 731(SU).

JULY: 732(ST), 733(ST), 734(ST), 735(ST), 736(ST), 737(PO), 738(PO), 739(PO), 740(PO), 741(PO), 742(PO), 743(HY), 744(HY), 745(HY), 746(HY), 747(HY), 748(HY)[c], 749(SA), 750(SA), 751(IA), 752(SA)[c], 753(SM), 754(SM), 755(SM), 756(SM), 757(SM), 758(CO)[c], 759(CO), 760(WW)[c].

AUGUST: 761(BD), 762(ST), 763(ST), 764(ST), 765(ST), 766(CP), 767(CP), 768(CP), 769(CP), 770(CP), 771(EM), 772(EM), 773(SA), 774(EM), 775(EM), 776(EM)[c], 777(ST), 778(ST), 779(SA), 780(SA), 781(SA), 782(SA)[c], 783(HW), 784(HW), 785(CP), 786(ST).

SEPTEMBER: 787(PO), 788(IR), 789(HY), 790(HY), 791(HY), 792(HY), 793(HY), 794(HY)[c], 795(EM), 796(EM), 797(EM), 798(EM), 799(EM)[c], 800(WW), 801(WW), 802(WW), 803(WW), 804(WW), 805(WW), 806(HY), 807(PO)[c], 808(SM)[c].

OCTOBER: 809(ST), 810(HW)[c], 811(ST), 812(ST), 813(ST)[c], 814(EM), 815(EM), 816(EM), 817(EM), 818(EM), 819(EM)[c], 820(SA), 821(SA), 822(SA)[c], 823(HW), 824(HW).

NOVEMBER: 825(ST), 826(HY), 827(HY), 828(ST), 829(ST), 830(ST), 831(ST)[c], 832(CP), 833(CP), 834(CP), 835(CP)[c], 836(HY), 837(HY), 838(HY), 839(HY), 840(HY), 841(HY)[c].

DECEMBER: 842(SM), 843(SM)[c], 844(SU), 845(SU), 846(SA), 847(SA), 848(SA)[c], 849(SM), 850(ST), 851(ST), 852(ST), 853(ST), 854(CO), 855(CO), 856(CO)[c], 857(SU), 858(BD), 859(BD), 860(BD).

VOLUME 82 (1956)

JANUARY: 861(SM1), 862(SM1), 863(PO1), 864(SM1), 865(SM1), 866(SM1), 867(SM1), 868(HW1), 869(ST1), 870(EM1), 871(HW1), 872(HW1), 873(HW1), 874(HW1), 875(HW1), 876(EM1)[c], 877(HW1)[c], 878(ST1)[c].

FEBRUARY: 879(CP1), 880(HY1), 881(HY1)[c], 882(HY1), 883(HY1), 884(IR1), 885(SA1), 886(CP1), 887(SA1), 888(SA1), 889(SA1), 890(SA1), 891(SA1), 892(SA1), 893(CP1), 894(CP1), 895(PO1), 896(PO1), 897(PO1), 898(PO1), 899(PO1), 900(PO1), 901(PO1), 902(AT1)[c], 903(IR1)[c], 904(PO1)[c], 905(SA1)[c].

MARCH: 906(WW1), 907(WW1), 908(WW1), 909(WW1), 910(WW1), 911(WW1), 912(WW1), 913(WW1)[c], 914(ST2), 915(ST2), 916(ST2), 917(ST2), 918(ST2), 919(ST2), 920(ST2), 921(ST2), 922(SU1), 923(SU1), 924(ST2)[c].

APRIL: 925(WW2), 926(WW2), 927(WW2), 928(SA2), 929(SA2), 930(SA2), 931(SA2), 932(SA2)[c], 933(SM2), 934(SM2), 935(WW2), 936(WW2), 937(WW2), 938(WW2), 939(WW2), 940(SM2), 941(SM2), 942(SM2)[c], 943(EM2), 944(EM2), 945(EM2), 946(EM2)[c], 947(PO2), 948(PO2), 949(PO2), 950(PO2), 951(PO2), 952(PO2)[c], 953(HY2), 954(HY2), 955(HY2)[c], 956(HY2), 957(HY2), 958(SA2), 959(PO2), 960(PO2).

MAY: 961(IR2), 962(IR2), 963(CP2), 964(CP2), 965(WW3), 966(WW3), 967(WW2), 968(WW3), 969(WW3), 970(ST3), 971(ST3), 972(ST3)[c], 973(ST3), 974(ST3), 975(WW3), 976(WW3), 977(IR2), 978(AT2), 979(AT2), 980(AT2), 981(IR2), 982(IR2)[c], 983(HW2), 984(HW2), 985(HW2)[c], 986(ST3), 987(AT2), 988(CP2), 989(AT2)[c].

---

c. Discussion of several papers, grouped by Divisions.

AMERICAN SOCIETY OF CIVIL ENGINEERS

OFFICERS FOR 1956

PRESIDENT
ENOCH RAY NEEDLES

VICE-PRESIDENTS

Term expires October, 1956:
FRANK L. WEAVER
LOUIS R. HOWSON

Term expires October, 1957:
FRANK A. MARSTON
GLENN W. HOLCOMB

DIRECTORS

Term expires October, 1956:
WILLIAM S. LaLONDE, JR.
OLIVER W. HARTWELL
THOMAS C. SHEID
SAMUEL B. MORRIS
ERNEST W. CARLTON
RAYMOND F. DAWSON

Term expires October, 1957:
JEWELL M. GARRELTS
FREDERICK H. PAULSON
GEORGE S. RICHARDSON
DON M. CORBETT
GRAHAM P. WILLOUGHBY
LAWRENCE A. ELSENER

Term expires October, 1958:
JOHN P. RILEY
CAREY H. BROWN
MASON G. PRICHARD
ROBERT H. SHERLOCK
R. ROBINSON ROWE
LOUIS E. RYDELL
CLARENCE L. ECKEL

PAST-PRESIDENTS
Members of the Board

DANIEL V. TERRELL
WILLIAM R. GLIDDEN

EXECUTIVE SECRETARY
WILLIAM H. WISELY

TREASURER
CHARLES E. TROUT

ASSISTANT SECRETARY
E. L. CHANDLER

ASSISTANT TREASURER
CARLTON S. PROCTOR

PROCEEDINGS OF THE SOCIETY

HAROLD T. LARSEN
Manager of Technical Publications

DEFOREST A. MATTESON, JR.
Editor of Technical Publications

PAUL A. PARISI
Assoc. Editor of Technical Publications

COMMITTEE ON PUBLICATIONS

SAMUEL B. MORRIS, *Chairman*
JEWELL M. GARRELTS, *Vice-Chairman*

ERNEST W. CARLTON
MASON G. PRICHARD
R. ROBINSON ROWE
LOUIS E. RYDELL